LOAD CARRYING CAPACITY OF ENCASED STONE COLUMN IN COMPACTED POND ASH BED

A thesis submitted in partial fulfillment of the requirements for the award of the degree of

Master of Technology

In

Civil Engineering

(Geotechnical Engineering)

By

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2014

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CERTIFICATE

This to certify that the thesis entitled "**Load Carrying Capacity of Encased Stone Columns in Compacted Pond Ash Bed**" being submitted by **Preetynanda Nanda** in the partial fulfilment of the requirements for the award of Master of Technology Degree in **Civil Engineering** with specialization in **GEOTECHNICAL ENGINEERING** at the National Institute of Technology, Rourkela is an authentic work carried out by her under my supervision and guidance.

To the best of my knowledge, the matter embodied in this report has not been submitted to any other university/institute for the award of any degree or diploma.

Place: Rourkela

Date:

Prof. Suresh Prasad Singh

Dedicated To

My caring and supportive father,

My loving and sacrificing mother

My beloved and saintly brother

ACKNOWLEDGEMENT

The satisfaction on the successful completion of any task would be incomplete without the mention of the people who made it possible whose constant guidance and encouragement crowned out effort with success.

I am grateful to the **Dept. of Civil Engineering, NIT ROURKELA,** for giving me the opportunity to execute this project, which is an integral part of the curriculum in M.Tech programme at the National Institute of Technology, Rourkela.

I would like to take this opportunity to express heartfelt gratitude for my project guide **Dr. S. P. Singh,** who provided me with valuable inputs at the critical stages of this project execution. I would also like to express my gratitude to Prof. N. Roy, Head Civil Engineering Department, Prof. C. R. Patra and Prof. S. K. Das for their help and constructive suggestions during the project work. I am also very thankful to all the faculty members of the department, especially Geo-Technical Engineering specialization for their constant encouragement during the project. I am also thankful to staff members of soil engineering laboratory especially Mr. Chamuru Suniani, Mr. Dilip Das and Mr. Narayan Mohanty for their assistance and co-operation during the course of experimentation.

The help and support received from my friends Pawan Kumar Chamling, B. Srikanth, Biraja Prasad Mishra and many more that made constructive comments and helped physically during the project work. Last but not the least; I am thankful to almighty, who kept me fit both mentally and physically throughout the year for the project work.

Above all, I would like to thank my parents, especially my brother Mr. Byomakesh Debata who has given me his unequivocal support, as always, for which my mere expression of thanks likewise does not suffice.

For any errors or inadequacies that may remain in this work, of course, the responsibility is entirely mine.

Preetynanda Nanda

SYNOPSIS

Pond ash deposits possess high compressibility, low bearing capacity so acres of land get wasted. Improvement of load carrying capacity of ash ponds will make them suitable for residential or commercial use. Stone or compacted stone columns is a technique of soil reinforcement that is frequently implemented in soft cohesive soils to increase the bearing capacity of the foundation soil, to reduce the settlement, and to accelerate the consolidation of surrounding saturated soft soil. The stress-strain behaviour of the granular column is governed mainly by the lateral confining pressure mobilized in the native soft soil to restrain bulging collapse of the granular column.

Several works have been done relating to find out the effectiveness of stone column on cohesive material, along with the effect of encasement and without encasement over the stone column. However no studies have been made to explore the effectiveness of stone columns in pond ash deposits. This study relates to the reinforcement of pond ash with stone column and possibility of utilizing abandoned ash pond sites for residential or commercial use.

The purpose of this work is to assess the suitability of reinforcing technique to improve the load carrying capacity of pond ash deposits through several laboratory model tests. This objective is achieved in three stages. In the 1st stage uniaxial compressive strength (UCS) tests were done on encased stone column and load carrying capacity of encased stone column (ESC) was evaluated by varying the slenderness ratio and relative density of compacted stone mass. It is found that load carrying capacity of stone column is directly proportional to the relative density of stone mass and inversely proportional to the slenderness ratio. In the 2nd series of test UCS test was conducted on encased stone column of slenderness ratio 2 and 3 reinforced with horizontal reinforcement made of galvanized iron (GI) and PVC mesh. In this case relative density of stone mass and spacing between the strips (d, 0.5d, 0.25d, where d is the diameter of SC) was taken as variables where d is the diameter of stone columns.

Placement of circular strip reinforcement increases the strength of SC. For the present test variable GI strip are found to be more effective than PVC strips. Further the improvement in load carrying capacity is more pronounced if the relative density of stone mass is higher. Circular GI strip placed at a spacing of 0.25d with relative density of stone mass 90% enhance the load carrying capacity by 11.44 times over the ESC. In the 3rd stage of experiment load test was conducted on pond ash bed reinforced with stone column and encased stone column. Further the SC and ESC were reinforced with horizontal GI and PVC grids placed at different positions. Installation of SC increases the load carrying capacity of pond ash bed by many folds, when SC is reinforced with horizontal reinforcement. Full encasement of SC reduces the bulging diameter and it carries more load than ordinary SC, embedded in pond ash bed. Further ESC reinforced with horizontal reinforcement carry much higher load than the other configurations of reinforcement tried in this test.

Further, the experimental results are compared with finite element analysis results using the software package PLAXIS and it is found that the ultimate load carrying capacity of stone columns for various reinforcement combinations are in good agreement with each other. Hence it is concluded that encasing of stone columns and reinforcing it with horizontal stiffer reinforcements will definitely improve the load carrying capacity of stone columns by many fold and it can be used in improving the load carrying capacity of pond ash deposits which are in general soft and compressible.

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INTRODUCTION

INTRODUCTION

Pond ash, generated during the combustion of coal for energy production, is an industrial by product that is recognized in India as an environmental pollutant. It is a collective mixture of fly ash and bottom ash in wet condition. A look at the statistics show that, out of India's total installed power capacity of 159,398.49 megawatts, 64.6 % comes from thermal energy and 80 % is obtained from coal. An estimated 65,000 acres of land is employed by ash ponds, indicating that more than 80 million metric tons is being stored in ponds or disposed of in landfills today. An estimated 225 million metric tons of fly ash that will be stored in more than 1.8 million acres of ponds in 2032. India produces about 112 million metric tons of fly ash annually. India’s dependence on coal as a source of energy shall continue in the next millennium and therefore fly ash management would remain an important area of national concern. Its indiscriminate disposal involves large volumes of land, water and energy. Pond ash deposit posses’ high compressibility, low bearing capacity so acres of land get unused. Pond ash can be stabilized using encased stone column to increase the bearing capacity and structures can be built on ash pond in a cost effective manner.

In an era of increasing land costs and growing population ash pond deposit have been a great headache for the technocrats, administrators, environmentalists and above for the civilization as it results in loss of agriculture production, grazing land and habitat as well as other land use impacts from diversion of large areas of land to waste disposal. Thousand acres of land occupied by pond ash deposits remains unused as it possess high compressibility and low bearing strength. The use of compacted stone columns as a technique of soil improvement is frequently employed in soft cohesive soils to increase the bearing capacity of the foundation soil, to reduce the settlement, and to accelerate the consolidation of the surrounding saturated soft soil. But very little work has been done on stone column for stabilization of ash ponds. Literature shows that compacted stone column as a stabilizing technique can be applied effectively in silty to fines and. Fly ash also comes in this range. So, in the present study an attempt has been made to study the effectiveness of encapsulated stone column in improving the bearing capacity of abandoned ash ponds.

In the present study three series of tests are conducted. In the first series UCS test is carried out on encased SC for different slenderness ratio (0.5, 1, 2, 3, and 4) and relative density of stone aggregates (30%, 60%, and 90%). In the second series of tests UCS test is conducted on encased SC reinforced with parallel strip of different stiffness at a spacing of d, 0.5 d , and 0.25d (where d is the diameter of SC) and varying relative density of stone aggregates and in the last series of test load test is conducted on stone column(SC), encased stone column (ESC) and stone column reinforced with different stiffness of circular strips placed at various position, installed in compacted pond ash bed.

1.1 ORGANIZATION OF THE THESIS

The thesis has been arranged in five chapters as discussed below:

Chapter 1: A brief report on the production, present disposal practices of pond ash is mentioned in this chapter along with the economical and environmental impact of fly ash. In addition to the different methods of ground improvement techniques adopted to improve the load carrying capacity of soft soil deposits are discussed in particular reference to use of stone column asground improvement method.

Chapter 2: In this chapter suitability of SC in enhancing the load carrying capacity of poor ground and various aspects of SC like its type, installation process, failure mode , advantages are briefly discussed. Literature review on SC has done and outlined in this chapter in the following categories:

- Numerical and Analytical Studies
- Theoretical Analysis
- Model tests
- Prototype/ Field test

Chapter3: The experimental work and methodology adopted is presented in this chapter. Index properties and geotechnical properties of pond ash and stone aggregates was determined and other properties of reinforcing material was obtained. Three series of tests were conducted on stone column. In the 1st series of test UCS test was conducted on encased SC with a variation in relative density of compacted stone aggregates and slenderness ratio of SC (0.5, 1, 2, 3 and 4). Another set of UCS tests were conducted on encased stone column having slenderness ratio of 2 and 3 with a variation in relative density of aggregates, stiffness of circular strip and spacing between the strips. In the third series of test load test on stone column and encased stone column installed in compacted pond ash bed was conducted with an alteration in stiffness of circular strip and its relative position in the SC.

Chapter4: The test results obtained by conducting various tests on pond ash, stone aggregates and reinforcing material in addition to the results achieved from the three series of test conducted on stone column are outlined in this chapter and discussed briefly.

Chapter5: The salient conclusions are reported.

CHAPTER 2
LITERATURE REVIEW

LITERATURE STUDY

2.1 Introduction

The Use of stone column as a ground improvement technique is of recent origin. Stone column consist of granular material compacted in long cylindrical hole.

Stone columns are extensively used to

- Improve the bearing capacity of poor ground to make it possible to use shallow foundation on the soil,
- Increase time rate of settlements, stiffness,
- Enhance shear strength of soil, drainage condition and environmental control.
- Reduce the settlement of structure,
- Reduce liquefaction potential of soft ground

The stone column technique is widely used to strengthen the ground so as to support various geotechnical facilities like embankments, oil tanks on poor ground, low-rise buildings, highway facilities, bridge abutments. The method is generally adopted in clayey soils. Further the load carrying capacity of SC can be improved by encasing the column with suitable geosynthetic. The advantages of encasing the column are as follows (Raithel et al. 2002, Alexiew et al. 2005)

- Additional lateral confinement.
- Making the SC to act as a semi rigid element enabling the load transfer to deeper depth.
- Preventing the lateral squeezing of stones in to the surrounding soft soils thereby minimizing the loss of stone.
- Enabling higher degree of compaction compared to the conventional SC.
- Promoting the vertical drainage function of the column by acting as a good filter.
- Preserving the frictional properties of aggregates.

- Increasing the shear resistance of SC (Ref Murugesan & Rajagopal 2009)

Various researchers have worked on stone columns and encapsulated stone column. Many numerical analyses, model tests, field tests, mathematical simulations are carried out to study the effects of stone columns on poor ground. However the design of stone columns till date is based on the empirical approach as the load settlement behavior of stone columns are influenced by a number of factors. The available literature on stone column is discussed in this chapter.

2.2 Methods of installation of stone column

Stone columns are commonly installed by water jetting or air jetting cylindrical vibrating probes into the ground to form holes, which are backfilled with gravel or crushed rock densified by the Vibratory probe as it is withdrawn from the ground. Water jetting is the part of the vibroflotation technology, which is often referred as vibro-replacement or wet, top feed method. Air jetting is part of the vibro-dispalcement or dry, top or bottom feed method. Sometimes, pre–augering is used to reduce the amount of ground displacement during the installation. The vibro-replacement method has less displacement and vibration disturbance to the surrounding ground than the vibro-displacement method, it has other environmental impacts due to slurry generated during the installation.

The Use of stone column as a ground improvement technique is of recent origin. The method is generally adopted in clayey soils. This can be treated as the extension of technique of densification of cohesion less soil by vibrofloat. Earlier stone columns were formed by vibrofloat but now they are also formed by forming a bore as in bored cast in situ concrete piles. The primary purpose of soil improvement by stone column technique is mainly to increase the bearing capacity of foundation soil and also to reduce post construction settlement. The method has been mainly used to improve subsoil below buildings, embankments. Stone columns are constructed using either Vibro-replacement or vibro-displacement methods.

2.2.1 Vibro-replacement method

Vibro-Replacement (stone column) technology extends the range of soils that can be improved by vibratory techniques to include cohesive, mixed and layered soils. Columns are constructed by the introduction of natural or recycled aggregate backfill to the tip of the vibrator.

The horizontal action of the vibrator displaces the stone laterally to form a dense structural element. As successive stone columns are formed, the intervening soil is densified, creating an integrated soil/stone matrix capable of supporting high, vertical loads.

Stone columns may be constructed by either the top feed or bottom feed methods. In the top feed method, the stone is introduced from the surface into the annulus created by the vibrator. In the bottom feed method, stone is delivered to the tip of the vibrator via an integrated hopper and tremie pipe assembly. Selection of the most appropriate method is dependent on site and subsurface conditions.

Changes occurred during the Vibro-Replacement process:

- Sand and gravel particles are rearranged into a more dense state (higher relative density DR)
- Mixed or fine-grained soils are radially displaced outwards and reinforced
- A significant increase is achieved in the horizontal to vertical effective stress ratio in the area surrounding each column
- Angle of internal friction is increased
- Limited settlement of the compacted soil mass is achieved
- Soil deformation moduli are increased

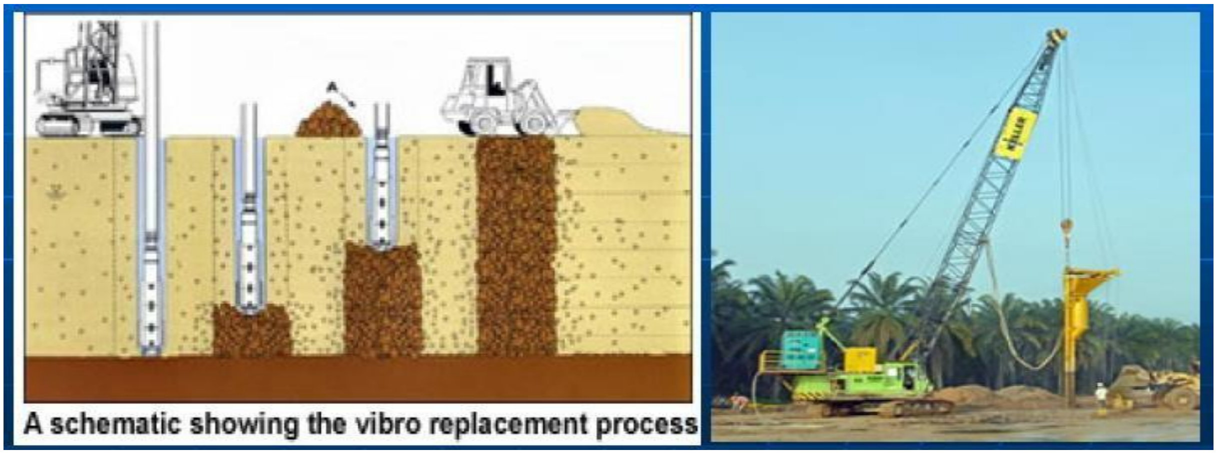


Fig-2.1 Vibro-replacement process

2.2.1.1 Wet top feed process

In the wet top feed process, the vibrator penetrates to the design depth by means of the vibrator's weight and vibrations, as well as water jets located in the vibrator's tip. The stone (crushed stone or recycled concrete) is then introduced at the ground surface to the annular space around the vibrator created by the jetting water. The stone falls through the annular space to the vibrator tip and fills the void created as the vibrator is lifted several feet. The vibrator is lowered, densifying and displacing the underlying stone. The vibro replacement process is repeated until a dense stone column is constructed to the ground surface.

Fig-2.2: Wet top feed process

2.2.1.2 Dry bottom feed process

The dry bottom feed process is similar except that no water jets are used and the stone is fed to the vibrator tip through a feed pipe attached to the vibrator. Pre drilling of dense strata at the column location may be required for the vibrator to penetrate to the design depth. Both methods of construction create a high modulus stone column that reinforces the treatment zone and densifies surrounding granular soils.

2.2.1.2.1 Advantages of the Bottom Feed Method

- Stone flow is mechanically controlled and automatically recorded
- Stone can be accurately delivered to precise depth levels
- Spoil return to the surface is minimal, resulting in lower stone consumption
- Mud handling is eliminated
- Surface loss of aggregate is reduced

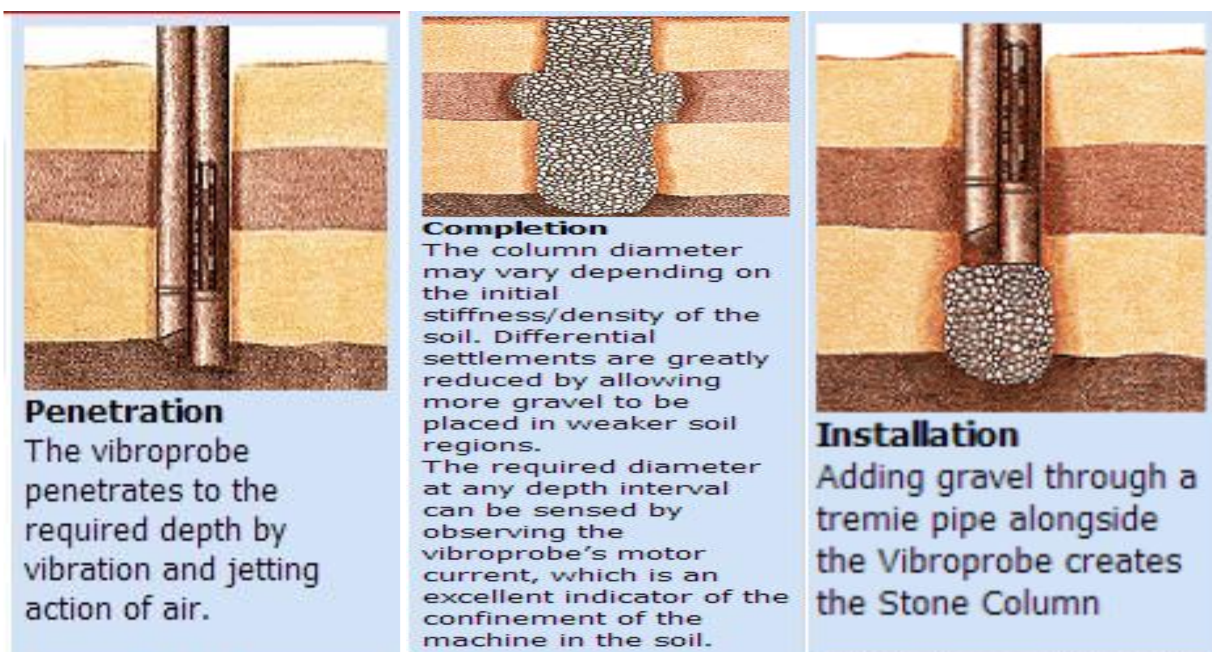


Fig-2.3: Dry bottom feed process

2.2.1.2.2 Results of Vibro-Replacement process :

- Increased bearing capacity, permitting shallow foundation construction or reduction of foundation sizes
- Transfer of vertical loads to reinforced or more competent soils at depth
- Increased composite shear strength for improved stability
- Significant increases in effective lateral stresses
- Settlement reduction under static and dynamic loading
- Near-elimination of differential settlement for large foundations
- Liquefaction mitigation
- Prevention of lateral spreading.

2.2.1.2.3 Suitability of Top feed method

- Water for flushing is available
- The working platform allows flushing water to flow towards settling ponds
- The soil type does not lead to problems handling the soil particles in the process water
- Space is available for a 5000 ft2 or suitably-sized sedimentation pond
- The installation crew has sufficient experience in the more demanding installation methodology

2.2.1.2.4 Suitability of bottom feed method

- Water for flushing is not available
- Washout of soil to the surface must be prevented due to potential soil contamination and/or difficulties in removing the washout

- Space is a limiting factor
- Very controlled column diameters are required
- The soil type does not allow for water flushing (i.e., peat, very soft soils)

2.2.2 Bored piling technique

This method has been developed in India has been gaining importance. A cased hole of required size is bored using conventional tools such as flap valve bailer and casing tube of required size. After the casing tube is driven to required depth, granular fill material is filled. Tube is withdrawn by short pass as required and granular fill compacted by rammer. The filling of the granular material, withdrawal of the casing tube and ramming of fill is so controlled as to have continuous column of stone column. Compaction is achieved by a rammer generally of 1.5 to 2tonnes and falling through a height of 1 to 1.5 m.

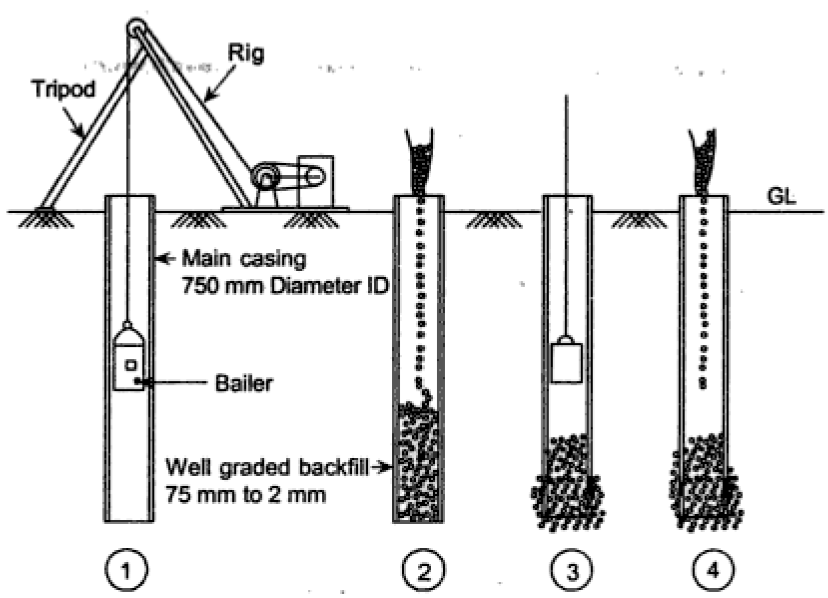


Fig-2.4 Stone column installation by ramming method

2.3 Design Concept

It is true that design of stone column is less understood but it is as empirical as the design of pile foundation. A stone column derives its support basically from lateral resistance provided by the surrounding soil to the expansion caused by bulging of the un-cemented stone column under the load.

The important parameters in estimating the capacity of stone column are

a. Angle of internal friction of the column material
b. Diameter of the stone column formed and
c. Un drained shear strength of surrounding soil
d. In-situ lateral stress in the soil
e. Radial pressure /deformation characteristics of the soil

The angle of internal friction depends on the material type, its gradation and shape and effectiveness of compaction. Generally angle of friction obtained is between 38^0 to 55^0. Higher angle can be adopted for the rammed stone columns that for the vibrated ones.

2.4 Suitable soils

The soil which does not respond to vibration alone is good for stone column. They are silty and clayey sands, silts, very fine sands, clays and some layered soils. The effectiveness of stone columns in different types of soil is given in Table 2.1.

2.5 Failure mechanism of stone column:

The possible modes of failure of stone columns are:

- Bulging Failure
- Pile Failure

- General Shear Failure

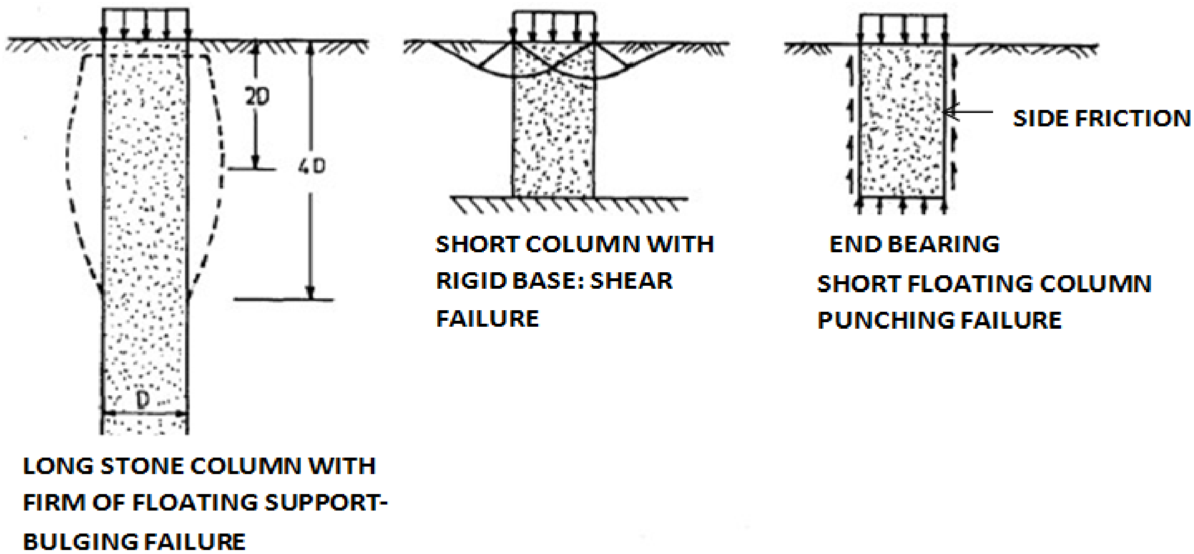


Fig-2.5 Failure mechanism of single stone column in a non-homogenous soft layer

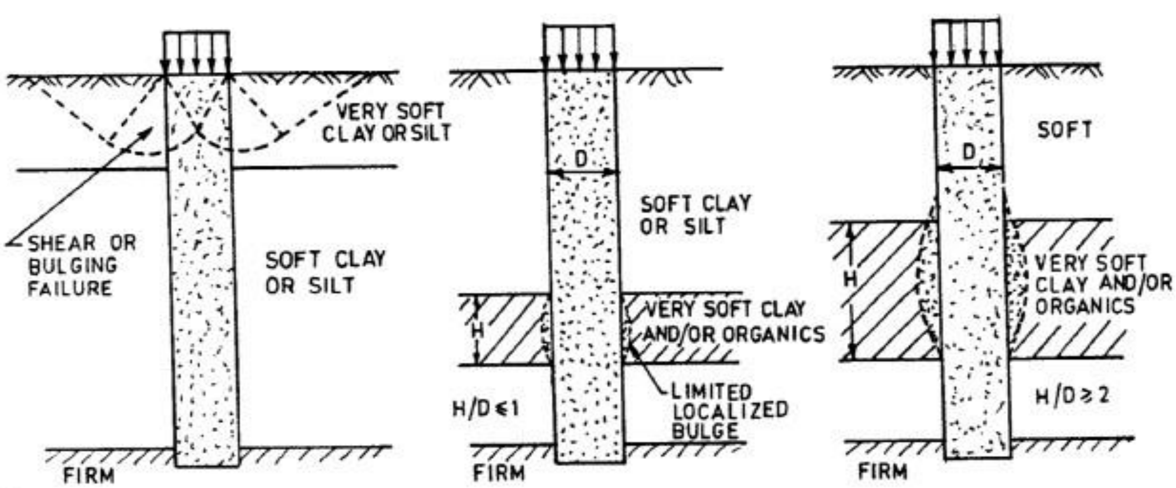


Fig-2.6 Failure mechanism of single stone column in a homogenous soft layer

Table-2.1 Effectiveness of stone column in different type of soil

Ground type	Relative effectiveness	
	Densification	Reinforcement
Sands	excellent	very good
silty sands	very good	very good
non plastic silts	good	excellent
Clays	marginal	excellent
mine spoils	excellent depending on gradation	good
dumped fill	good	good
Garbage	not applicable	not applicable

2.6 Advantages of stone column

- Stone Columns are designed to reduce settlements of compressible soil layers in order to be able to build most structures with shallow footings and slab-on-grades on very soft soil;
- When applicable, their draining characteristics result in an increase in the time rate of consolidation settlement in soft cohesive soil;
- Because they are made of compacted granular material, no curing period is necessary and no cut-off to the shallow footing grades are required as the excavation of the footing can immediately follow the installation of the stone columns down to the required elevation;
- Stone Columns are also well-adapted to the mitigation of liquefaction potential thanks to the combined effect/advantage of their draining potential and the increase of shear strength and stiffness of the improved soils.
- High production rates.

2.7 Application of stone column

- Industrial warehouses and commercial buildings;
- Condominium, apartment buildings, town houses and single-family residential developments;
- Reclaimed platforms (harbors, container terminals);
- Sewage treatment plants;
- Railway and roadway embankments;
- Retaining walls;
- Liquefaction mitigation and building support in seismic areas.

2.8 load settlement behavior of stone column

Various researchers have worked on stone columns. These works mainly focus on evaluation of load carrying capacity and settlement analysis of soft ground reinforced with stone columns. All these works can be grouped under the following sub headings:

- Numerical and Analytical Studies
- Theoretical Analysis
- Model tests
- Prototype/ Field tests

2.8.1 Numerical and Analytical Studies:

1. **Castro and Sagaseta (2011)** presented an analytical solution in which the soft soil is treated as an elastic material and the column as an elastic-plastic material using the Mohr-Coulomb yield criterion with a constant dilatancy angle. An elasto plastic

behaviour is considered for the encasement. The results are in agreement with numerical analyses.

2. **Mandal andDutta(2012)** performed axi-symmetric numerical analysis using finite element software PLAXIS 2D on end bearing stone columns without and with geogrid encasements. Variation in axial stiffness and length of encasement is done to analyse their effects on the behaviour of reinforced soft clay foundation. The load carrying capacity increases as the length of encasement increases. Increase in the stiffness of the encasement improves the behaviour of the encased stone column.

3. **Pulko et al. (2011)** analytical model is developed in which column is considered as an elasto-plastic material with constant dilatancy, soil as an elastic material and geosynthetic encasement as a linear elastic material. The result shows the influence of key parameters and provides a basis for rational predictions of settlement response for various encasement stiffness, column arrangements and load levels.

4. **Kaliakin et al. (2012)** described the results from a series of three dimensional finite-element analyses that are performed to simulate the behaviour of a single geo-synthetic encased stone column in soft clay. A comparative study is performed to simulate the behaviour of a dense and granular soil within the encasement.

5. **Indraratna et al. (2013)** developed a numerical and analytical solution for ascertaining the response of column reinforced soil on the basis of the equal strain hypothesis. Finite difference method is used to analyse the response of stone column-reinforced soft soil under embankment loading, adopting free strain approach and considering both arching and clogging effect. The proposed model predicts the

dissipation of excess pore water pressure and the resulting consolidation settlement with time.

6. **Khabbazian et al.(2011**) conducted three dimensional finite element analyses of geosynthetic encased columns in soft clay using three common functional form of the hyperbolic model for the encased granular material.

7. **Marto et al.(2013)** carried out numerical analysis for simulating the behaviour of un-encased and geogrid encased stone column in soft clay and presented the assumptions, procedures and results of the analysis. Settlement and bearing capacity of stone column and geogrid encased stone column are compared using various diameters of stone column.

8. **Malarvizhi and Ilamaruthi (2008)** carried out finite element analysis on encapsulated stone column using appropriate material models. The geogrid is modelled using geogrid element and Mohr-Coulomb model is used for stone column material. The stress-strain behaviour of the composite encased stone columns is predicted from FEM and compared with the experimental values.

9. **Ambily and Gandhi (2006)** determined the actual stress intensity on the stone column and soil using Finite Element Analysis(PLAXIS). Sand pad is provided at the surface for drainage and the effect of sand pad thickness on load sharing between column and soil is studied by the analysis for both flexible and rigid loading condition.

2.8.2 Theoretical Analysis:

10. **Tandel et al.(2012)** discussed the key consideration for the general use of encased stone column, insights for design and construction and compiled the latest research developments. It is found that the performance of encased stone column of smaller diameter is superior to that of larger diameter stone columns for the same encasement.

11. **Rao et al.(2013)** reviewed all the developments made on the use of granular anchored pile footing installed in expansive soils in terms of their efficiency in controlling the swell-shrink behaviour of footings resting on these soils. Possible use of geosynthetic encasement to granular pile is discussed here.

12. **Mokhtari and Kalantari (2012)** presented the installation methods, design and failure modes of stone column.

13. **Zhang et al. (2012)** developed a theoretical solution for calculating the consolidation of foundations reinforced by geosynthetic encased stone columns. The influence of geosynthetics on the consolidation of composite foundation is analysed.

14. **Deb et al.(2012)** developed an Evolutionary Genetic Algorithm NSGA(Non-Dominated Sorted Genetic Algorithm) for analysing the stability of geosynthetic reinforced embankments resting on stone column and used this to locate the critical surface and to optimize the corresponding factor of safety under various conditions.

15. **Deb et al.(2010)** presented a mechanical model to predict the bahavior of geosynthetic reinforced granular fill resting over soft soil improved with group of

stone columns subjected to circular or axi-symmetric loading in which saturated soft soil, granular fill, geosynthetic reinforcement and stone columns are idealized by spring-dashpot system, Pasternak shear layer, rough elastic membrane and stiffer springs respectively. The results obtained by the model are compared with the results obtained by the laboratory model tests.

2.8.3 Model tests:

16. **Murty et al. (2011)** presented results from experimental studies conducted on stone columns installed in the marine clay when subjected to cyclic loads done in the laboratory. Unit cell approach is adopted for testing of single stone column. Static and cyclic load tests are conducted on stone column and behaviour is studied.

17. **Sharma et al. (2012)** performed tests on stone columns by providing reinforcement in the form of horizontal strips of geosynthetic at different spacing over different column length and as encasement over the full column length. Encasement over the full column length gives higher failure stress as compared to the encasement over the top one-half column length for both floating and end bearing columns. It is observed that the best configuration of geotextile strips is the placement of the strips over full column length at d/2 spacing.

18. **Beena and Shukoor (2012)** studied the behaviour of stone column in which a portion of the broken stone is replaced by locally available material like rice husk. Stone column provides a drainage path to the water confined in the clay and rice husk degrades the consolidation of clay. It shows that partial replacement of stone with rice husk does not affect the performance of stone column.

19. **Raju et al. (2012)** studied the load versus settlement response of stone column and geotextile encased stone column .Load tests are performed on black cotton soil stabilized with four columns arranged in square pattern and reinforced stone column having different L/D and S/D ratios. The settlement in reinforced stone column is less than unreinforced stone columns and it decreases with increasing stiffness of the encasing material.

20. **Ayothiraman and Soumya (2011)** used shredded waste tyres as an alternative material to stone aggregates in construction of stone columns. It is concluded that waste tyre chips can be used as partial replacement of stone aggregates up to about 60% in stone columns.

21. **Murugesan and Rajagopal (2010)** performed tests on the qualitative and quantitative improvement of individual load carrying capacity of encased stone column. Load tests are performed on single and group of stone columns with and without encasement. It is found that load carrying capacity of the stone column increases due to encasement. The increase in load capacity depends on the modulus of encasement and the diameter of the stone column.

22. **Gniel and Bouazza (2009)** discussed the results obtained from series of model tests conducted to investigate the behaviour of geogrid encased stone column. Length is varied to see the behaviour of partially and fully encased stone column. It is found that for a partially encased column there is a steady reduction in vertical strain occurs due to increase in encased length for both isolated columns and group columns. For a

fully encased column there is a increase in column stiffness and decrease in column strain with strain reduction in the order of 80%.

23. **Sharma and Phanikumar (2005)** presented the heave behaviour of expansive clay reinforced with geopiles that are vertical cylindrical cells made of geogrid and filled with geomaterials. Effect of diameter of geopile and the type of the fill material on heave response have been investigated. It is found that heave decreases with increasing diameter of the geopile and particle size of the fill material. In the case of a group of geopiles, spacing between the geopiles is varied and its effect on heave is studied. Also heave decreases with closer spacing of geopiles.

24. **Najjar et al. (2010)** evaluate the degree of improvement of mechanical properties of soft clay using sand columns. The height of column, type of column (encased, nonencased) and effective confining pressure is varied. Test results indicated that sand columns improve the undrained strength.

25. **Deb et al.(2011)** presented the results from a series of laboratory model tests on un reinforced and geogrid reinforced sand bed resting on stone column improved soft clay. It is observed that load carrying capacity of soft soil is increased due to the placement of sand bed over stone column improved soft clay and bulge diameter of stone column reduces as the depth of bulge increases.

26. **Isaac and Girish(2009)** studied the performance of stone column using five reinforcement materials like stones, gravel, river sand, sea sand and quarry dust. Load versus settlement response is determined. It is found that there is no significant difference in the load deformation behaviour of stone columns using river sand and sea sand. Finite element analysis is also carried out using PLAXIS.

27. **Ambily and Gandhi (2007)** carried out an experimental study on behaviour of single column and group of seven columns by varying the parameters like spacing between the columns, shear strength of soft clay and loading condition. Finite Element Analysis (PLAXIS) is also performed using 15-noded triangular elements and results obtained are compared with the experimental results.

2.8.4 Prototype/ Field tests:

28. **Poorooshasb and Meyerhof (1997)** studied the efficiency of end bearing stone column and lime column in reducing the settlement of a foundation system. The influence of various factors like column spacing, weak soil properties, properties of the granular medium used in constructing the column, in situ stresses caused by the installation technique, the depth of bed rock relative to the tip of the columns and magnitude of the load carried by the supported raft foundation is examined.

29. **Lee et al. (2008)** investigated the improvement in load carrying capacity and reduction in bulging of a geogrid encased stone column using field load tests. It is found from the load test that geogrid encased stone column have much higher load carrying capacity and less lateral bulging compared to conventional stone columns.

CHAPTER-3

EXPERIMANTAL WORK AND METHODOLOGY

EXPERIMANTAL WORK AND METHODOLOGY

3.1INTRODUCTION

Load settlement behavior of stone column embedded in compressible ground is studied using numerical methods, analytical studies, model tests and prototype/ field tests. In the present test condition UCS test was conducted on encased stone column having different slenderness ratio (0.5, 1, 2, 3 and 4) and a variation in relative density of compacted stone aggregates. Further UCS test was carried out on ESC having slenderness ratio of 2 and 3 reinforced with GI strip or PVC strip with an alteration in position of strips. In addition to this the suitability of stone columns in improving the load carrying capacity pond ash deposits were examined through a series of model tests. This chapter outlines experimental work undertaken, the methodology adopted and the salient test results.

3.2 MATERIALS USED

3.2.1 Pond Ash

The pond ash was collected from NTPC Kanhia, Odisha. Pond ash was dried in the oven at 105˚C-110˚C and kept in an air tight container for further use. The physical properties were determined and presented in Table-3.1

3.2.2 Stone Aggregates

Stone aggregates were obtained from the local crushing unit of Rourkela. All these aggregates were washed and oven dried at the temperature of 110^{0}C. The stone aggregates were stored in an airtight container for subsequent use. The size of the stone aggregates varies from 2mm to 6mm. The physical properties were determined and are presented in Table-3.1.

Table-3.1 Physical properties of pond ash and stone aggregates

Physical parameter	**Pond ash**	**Stone aggregate**
Colour	Grey	Light grey
Shape	Rounded/sub rounded	Rounded/sub rounded
Size	-	2mm-6mm
Uniformity coefficient	8.33	-
Coefficient of curvature	0.75	-
Specific gravity	1.97	2.67
$\rho_{d,max}$ & $\rho_{d,\ min}$		1.57 g/cc & 1.42g/cc

3.2.3 Encasing Materials

PVC net having 1mm aperture was used for encasing the SC. GI strip of 1mm aperture and PVC net were used as circular parallel strips for reinforcing action. Tensile strength test was carried out on the encasing materials and the values obtained are presented in Table-3.2

3.3 TEST PROGRAMME AND METHODOLOGY

Three series of tests were conducted on stone column. In the first series of test UCS test was carried out on encased SC for different slenderness ratio (0.5, 1, 2, 3, and 4) and relative density (30%, 60%, and 90%). The diameter of SC was taken as 100mm. in the second series of test UCS test was conducted on SC having slenderness ratio 2 and 3 with a variation in relative density (30%, 60%, and 90%), stiffness of circular strips, and spacing between the circular strips (d, 0.5d, and 0.25d), where d is the diameter of SC. In the third series of test load test was conducted on SC and ESC inserted pond ash bed. The diameter of SC was taken as 50mm and length as 250mm. Further the SC and ESC are reinforced with circular strips of different stiffness and spacing between the strips. The details of the tests conducted and the experimental procedure are specified below.

3.3.1 Determination of Index Properties

3.3.1.1 Determination of Specific Gravity

Specific gravity of pond ash and stone aggregates was found as per IS: 2720 (Part III) and obtained as 1.97 and 2.67 respectively.

3.3.1.2 Determination of Maximum and Minimum Density of Stone aggregates

Minimum and maximum dry density of stone aggregate was determined as per IS-2720 part (14). Minimum dry density was determined by filling the standard mold with aggregates to their loosest state. Maximum dry density was determined with respect to their densest state using vibrating table and putting a surcharged weight over it, as per provisions of IS-2720 part (14). The maximum and minimum dry densities of stone aggregate obtained are presented in Table-1.

3.3.1.3 Grain size distribution for Pond ash

Grain size distributions pond ash was conducted as per IS: 2720 part (IV) for coarse fractions and hydrometer analysis was conducted for finer particles. Grain size distribution curve of pond ash is presented in Fig-3.1. Coefficient of uniformity (C_u) and coefficient of curvature (C_c) of pond ash are presented in Table-3.1.

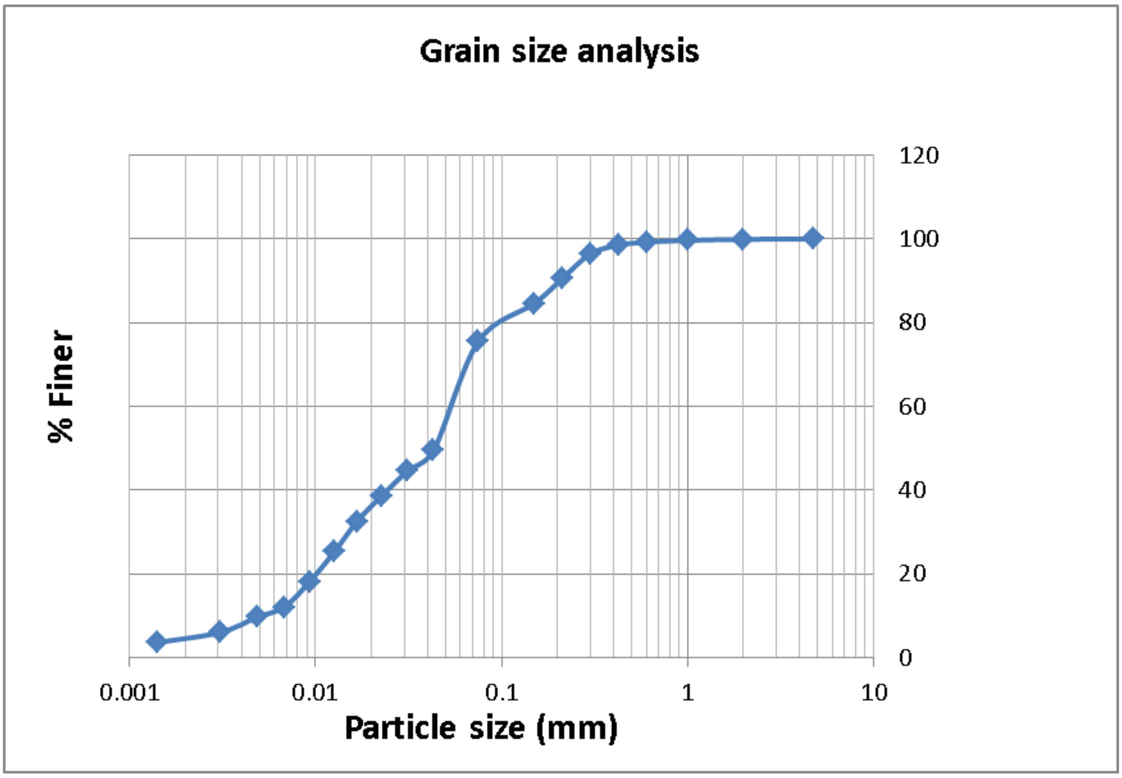


Fig-3.1 Grain size distribution of pond ash

3.3.2 Determination of Geotechnical Properties.

3.3.2.1 Determination of OMC & MDD of pond ash

Light compaction test was carried out on pond ash as per IS: 2720 (Part VII). The OMC & MDD of pond ash is found out to be 25% & 1.17g/cc respectively. Heavy compaction test was conducted on pond ash and the OMC & MDD was obtained as 22% & 1.28g/cc. Plot between water content(%)-dry unit weight(kN/m^3) for light compaction test and heavy compaction test is plotted in Fig-3.2 and 3.3 respectively.

For preparation of pond ash bed compaction test was carried out on standard proctor mould. Sample was placed in three equal layers and each layer was given 5 no of blows with hammer. Maximum dry density and optimum moisture content from the test was obtained as 1.15 g/cc and 28% respectively. This OMC and MDD were used for the preparation of pond ash bed. Graph between water content (%)-dry unit weight (kN/m^3) for pond ash compacted in a standard proctor mould with 5 no of blows is plotted in fig-3.4.

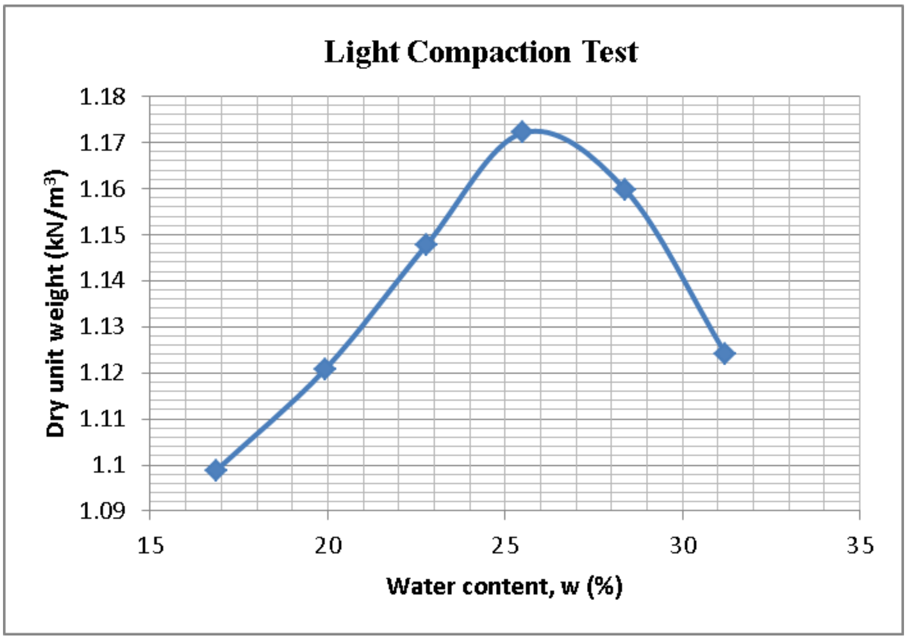


Fig-3.2 Water content- Dry unit weight of pond ash for Light compaction test

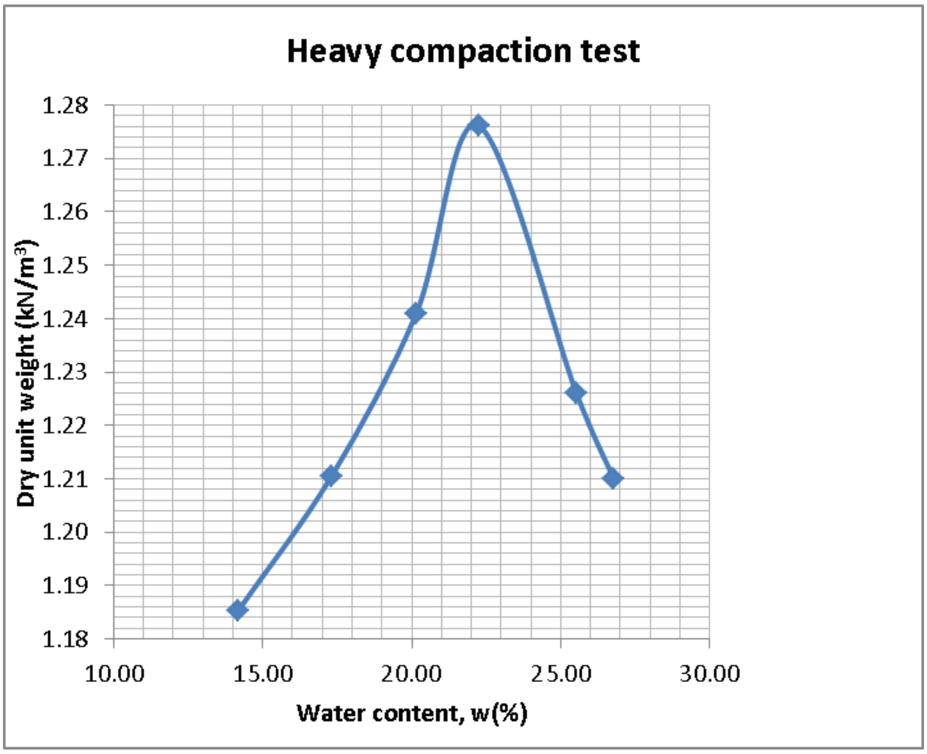


Fig-3.3 Water content- Dry unit weight of pond ash for Heavy compaction test

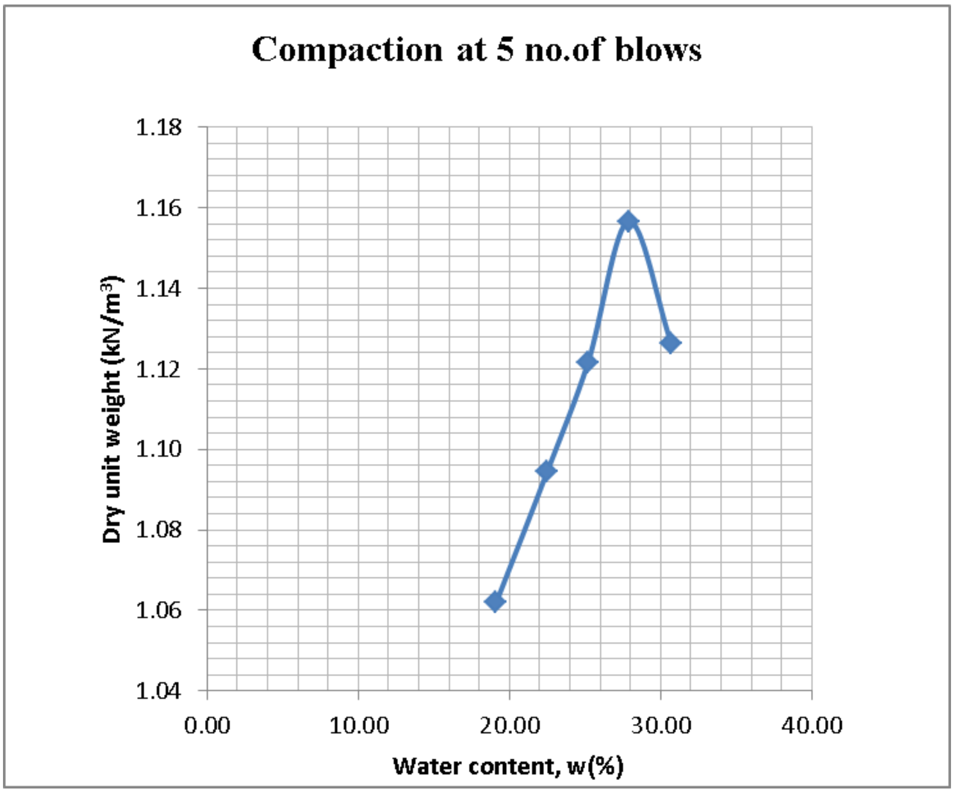


Fig-3.4 Water content- Dry unit weight of pond ash compacted at 5 no of blows

3.3.2.2 Determination of Shear parameters of Pond ash

The shear parameters of pond ash sample compacted to their corresponding OMC and MDD were determined as per IS: 2720 (Part 13). These specimens were of size 60mm×60mm×25mm deep and sheared at a rate of 1.25 mm/minute. The shear strength parameters of the compacted specimens were determined from normal stress versus shear stress plots and it is given in fig-3.5. Cohesion and angle of internal friction was found to be 0.11kg/cm^2 and 34° respectively.

3.3.2.3 Determination of Shear parameters of stone aggregates

Direct shear test was conducted on stone aggregate compacted at 30% and 60% relative density. The test was conducted on a mould of size 150mm×150mm×75mm and the specimen was sheared at a rate of 1.25mm/minute. The shear strength parameters of the compacted specimens were determined from normal stress versus shear stress plots and it is given in fig-3.6 and 3.7. Angle of internal friction was found to be 44° and 47° respectively for stone compacted at 30% and 60% relative density.

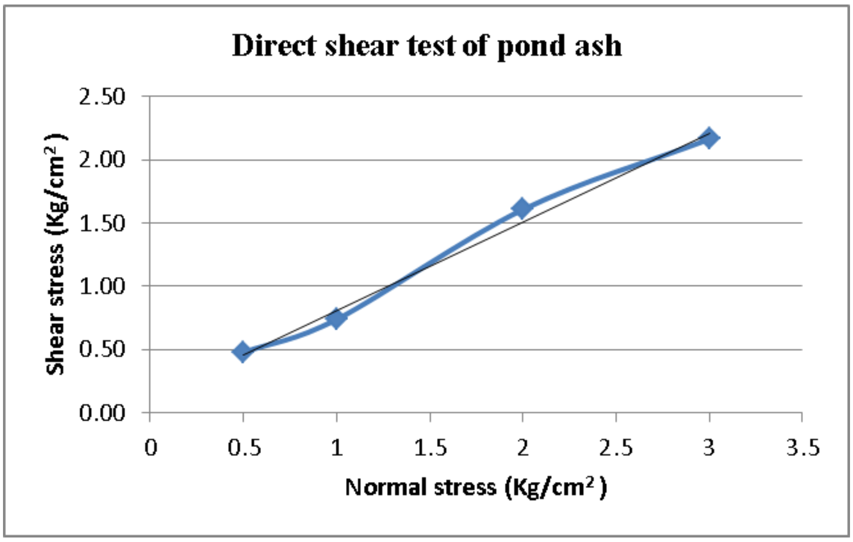


Fig-3.5 Normal stress- Shear stress of Pond ash

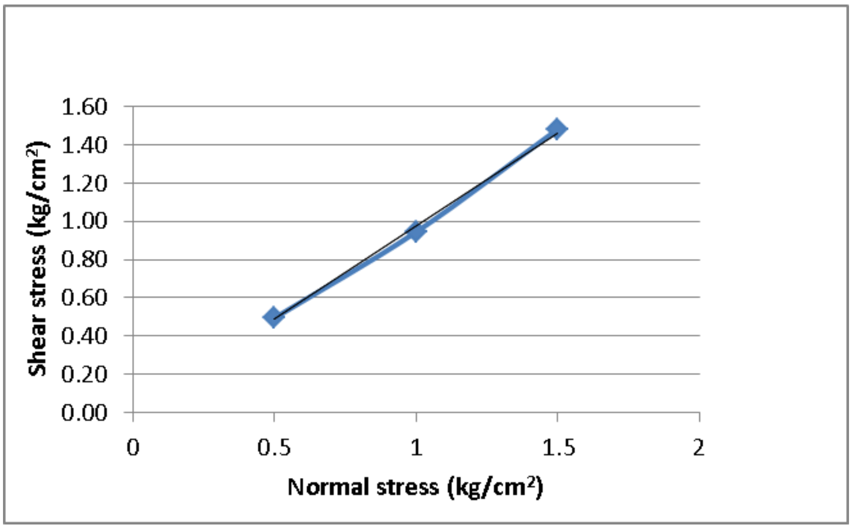


Fig-3.6 Normal stress- Shear stress of stone aggregate compacted at 30% relative density

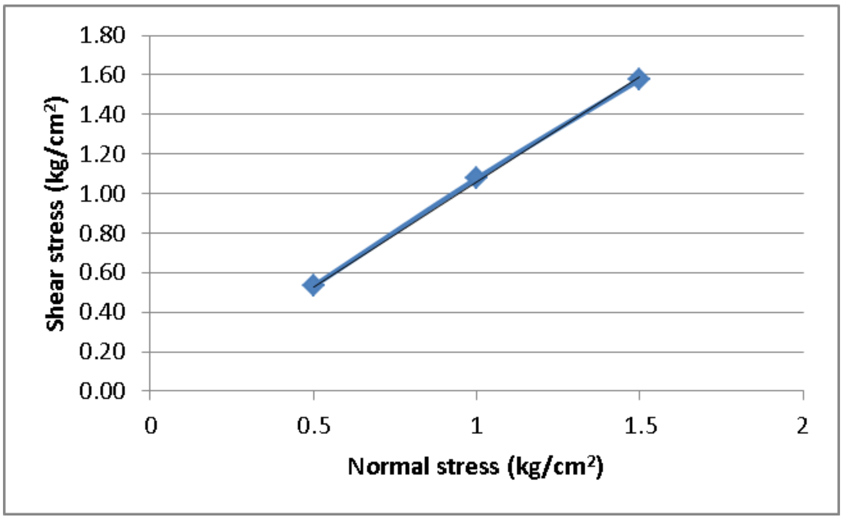


Fig-3.7 Normal stress- Shear stress of stone aggregate compacted at 60% relative density

3.3.3 Determination of Tensile strength of Reinforcing Material

Tensile strength test was carried out on reinforcing materials. The properties obtained are given in Table.3.2

Table-3.2 Properties of reinforcing material

Sample id	GI sheet	PVC mesh
Displacement at peak (mm)	5.368	82.42
Strain at peak (%)	15.34	235.5
Load at peak (kN)	0.6029	0.0210
Stress at peak (Mpa)	25.37	0.8030
Displacement at break (mm)	[illegible]	[illegible]
Strain at break (%)	21.72	364.0
Load at 0.2% yield (kN)	0.288	0.0085
Stress at 0.2% yield (Mpa)	12.15	0.3250
Young's modulus	1082	39.16

3.4 UCS TEST ON ENCASED STONE COLUMN

3.4.1 UCS Test on SC of different Slenderness Ratio

UCS test was conducted on stone column as per IS: 2720 (Part X). The test was carried out on encapsulated SC to study the effect of slenderness ratio and relative density of stone aggregates on load carrying capacity of SC. The diameter of SC was taken as 100mm. Cylindrical shape was stitched using the reinforcing material PVC having 1mm larger diameter than the SC diameter in order to achieve the slenderness ratio of 0.5, 1, 2, 3, and 4. Calculated amount of stone mass was taken and compacted with a tamping rod of 70mm diameter so as to meet the desired relative density of 30%, 60%, and 90%. Solid circular plate was placed below the cylindrical encasement, and then required mass of stone aggregate was poured inside it and compacted with a tamping rod to achieve the preferred relative density. The compacted SC was tested under the UCS testing machine at a rate of 1.25mm/min and the load corresponding to various deformations was obtained. The stress-strain graph was plotted. Fig-3.8 shows the ESC under the loading frame and fig-3.9 shows the accessories required for making SC.

Fig-3.8 Encased SC under the testing machine

Fig-3.9 Tamping rod used to compact stone aggregates

3.4.2 UCS test on Encased SC Reinforced with Circular Strips

A cylindrical profile was stitched with PVC mesh having length and diameter slightly larger than the length and diameter of SC. The diameter of SC was taken as 100mm and length was varied as 200mm and 300mm. Circular strips of diameter 100mm was cut from PVC net and GI sheet, which is used as horizontal reinforcement for the SC. The circular strips were provided inside the SC with a varying spacing of d, 0.5d, and 0.25d, where d is the diameter of SC.

The cylindrical profile was taken and the calculated stone mass was placed inside it. The mass of stone aggregates were taken so as to achieve the relative density of 30%, 60%, and 90%. In order to maintain the spacing of d, 0.5d, and 0.25d for SC of slenderness ratio equals to 2, the no. of strips required are 1, 3, and 7 respectively. SC of slenderness ratio equals to 3 required 2, 5, and 11 no.of strips for d, 0.5d, and 0.25d spacing respectively. After putting the desired stone mass up to the required height it was compacted with the tamping rod. Then the circular strip was placed over the compacted stone mass. Again the calculated stone mass was placed over the circular strip and compacted. The process was continued to accomplish the chosen vertical spacing between the circular strips and slenderness ratio of SC. Then test was conducted on encased SC reinforced with circular strips placed at altered vertical spacing. The compacted SC was tested under the UCS testing machine at a strain rate of 1.25mm/min and the load corresponding to various deformations was obtained. The stress-strain graph was plotted. Fig-3.11 shows the schematic diagram of SC having slenderness ratio 2 and 3 reinforced with horizontal strip for different configuration of spacing.

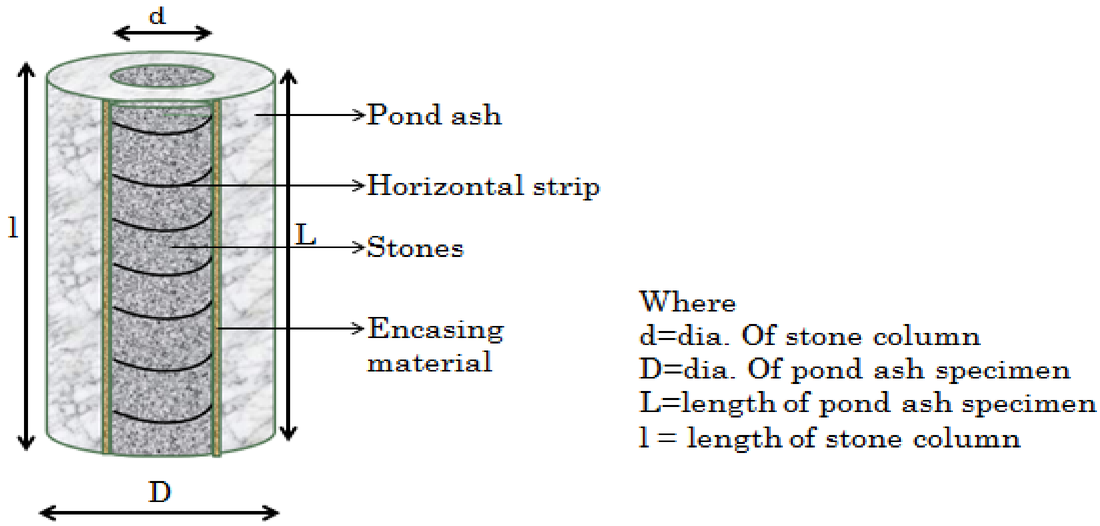


Fig.3.10 Schematic diagram of stone column embedded in pond ash

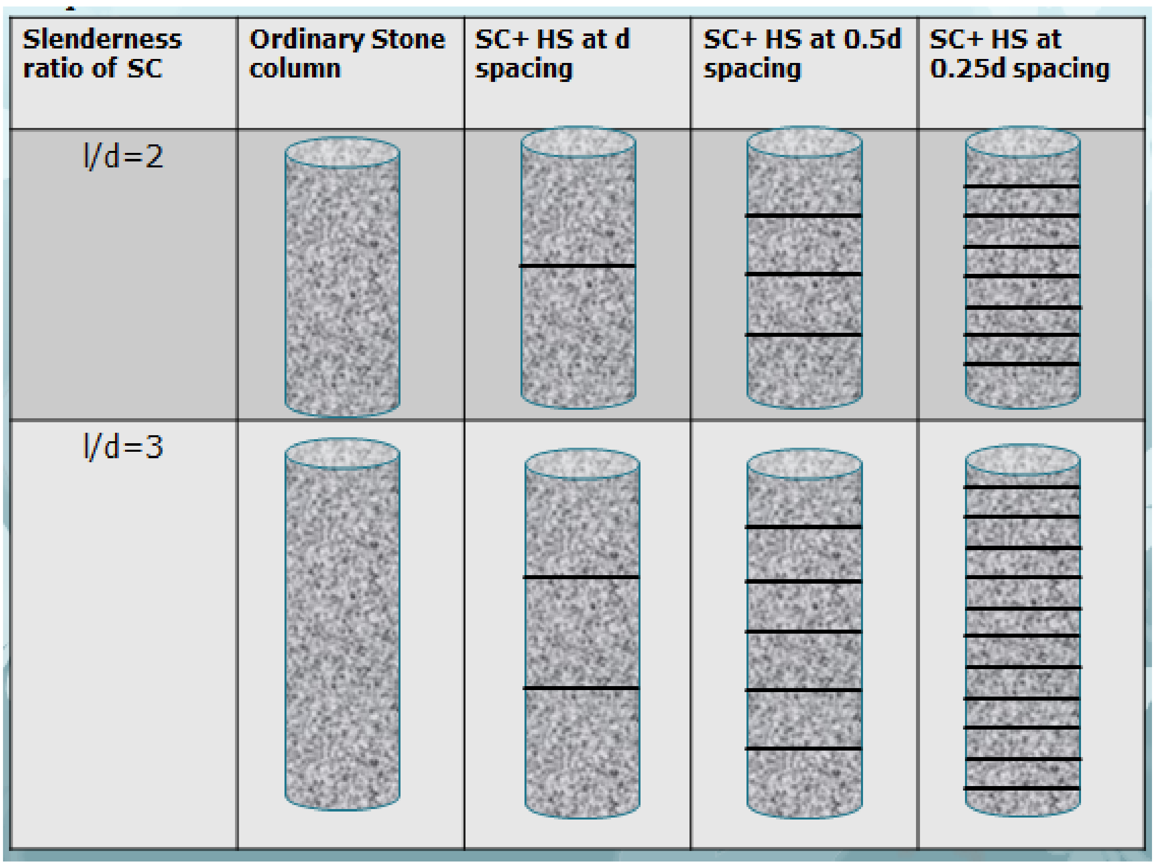


Fig-3.11 Position of reinforcing material in a SC having slenderness ratio of 2 & 3

3.4.3 Load test on SC and ESC embedded in pond ash bed

A cylindrical tank of diameter 256mm and height 305mm was taken for footing load test on pond ash bed reinforced with ordinary stone column and encased stone column. The footing is of 50mm diameter and 5mm thickness. Pond ash was compacted on the standard proctor mold in three layers and each layer was compacted with 5 numbers of blows. The OMC and MDD obtained from the above test condition were taken for preparing the pond ash bed. The MDD and OMC are obtained as 1.16g/cc and 28%. Pond ash used for preparing the bed was oven dried at 108˚C and kept in an air tight container for further use. Desired amount of oven dried pond ash was taken and 28% water was added to it and mixed thoroughly. Three marks were put on the inner side of the tank. The mixture obtained was distributed in to three equal parts. The whole mass was compacted in the tank in three equal layers by means of hydraulic jack. For making stone column a hollow steel pipe of outer diameter 50mm was taken. In the compacted pond ash bed the center was determined and the hollow pipe was placed centrally over it. Pipe was inserted into the bed by the application of hammer blows. Pipe was taken out carefully when it reached the bottom of the tank and the material inside the hollow pipe was removed. The SC for this case has a diameter of 50mm and length 250mm. Stone aggregates taken for this case has size varying from 2mm to 6mm. Desired amount of stone mass was taken to achieve a density of 1.51g/cc. Stone aggregates were poured inside the hole with a funnel in 5 equal layers and each layer was compacted with a tamping rod of 30mm diameter to the certain height in order to achieve the density. The footing was placed centrally over the stone column. In all the cases, the prepared test bed tank along with SC was placed under a loading frame. Loading was applied through a footing resting on the prepared pond ash bed and resistance offered by the test bed with or without SC was measured with the help of proving ring. Load was applied up to 50mm settlement at a strain rate of 1.25mm/min. Load corresponding to various deformation was obtained and load-settlement curve was plotted. Fig-3.10 shows the schematic diagram of SC embedded in pond ash bed. Fig-3.12 shows the picture of SC embedded in compacted pond ash bed.

For encased stone column cylindrical profile was made with a diameter of 50mm and length 250mm. After the extracting material form the hole prepared on pond ash bed the cylindrical profile was inserted very carefully so that it will have contact with the surrounding mass. Calculated mass of stone aggregates were placed inside the hole and compacted to achieve the required density.

For ordinary stone column reinforced with horizontal strips, circular strips of 50mm diameter were cut from PVC mesh and GI sheet. Various configurations were taken to place the circular strip inside the stone column. Required amount of stone was put and compacted to attain the desired density. Strip was placed at 0.5d, d, 2d, 3d and 4d from the top. 2, 3, 4 number of strips were placed at a spacing of 0.5d from top up to a height of 5mm, 7.5mm and 10mm respectively.

3.4.4 Method to obtain the bulged shape of SC

To observe the bulging of SC after testing, thin slurry of cement and plaster of paris was added in the stone column without disturbing the column and allowed to get sufficient strength so that even after removal of pond ash surrounding the stone column, the shape of the tested SC remained unaffected. After removal of the SC from the pond ash bed the bulge depth and bulge diameter of the columns were measured. Deformed shape of SC and ESC are plotted in fig-3.13 and 3.14.

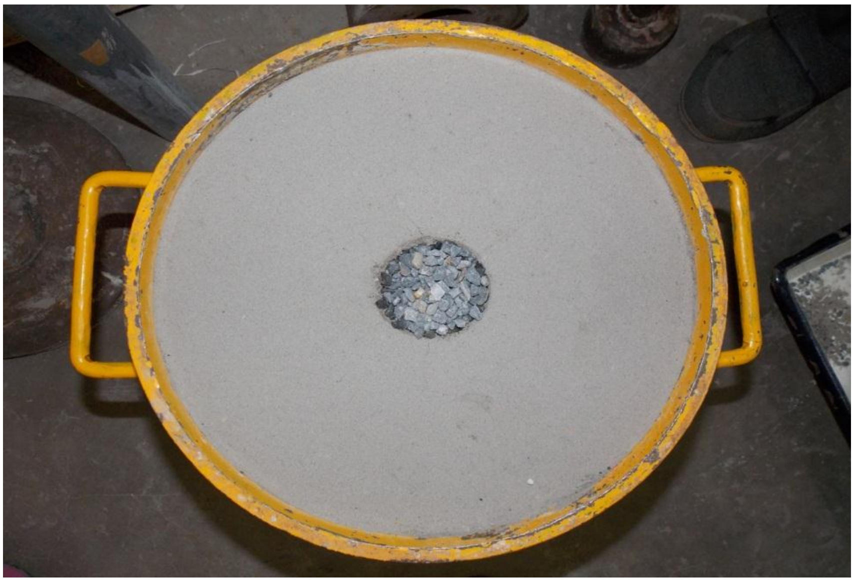


Fig-3.12 Stone column embedded in compacted pond ash bed

Fig-3.13 Deformed shape of SC   Fig-3.14 Deformed shape of ESC

CHAPTER-4
TEST RESULTS AND DISCUSSION

4.1 INTRODUCTION

Pond ash possesses very low bearing capacity and it is compressible in nature. The load carrying capacity of pond ash can be improved by the installation of stone column (SC) and encased stone column (ESC). In the present test condition series of tests were conducted on encased SC to know its behavior. In the 1st series of test behavior of ESC was studied by varying the slenderness ratio as 0.5, 1, 2, 3 and 4 along with a change in relative density as 30%, 60% and 90%, under uniaxial compressive loading. For the 2nd series of test compressive loading was done on ESC having slenderness ratio of 2 and 3 reinforced with horizontal reinforcement made of GI and PVC strip with a variation in relative density of compacted aggregate and spacing between the strips. The spacing of strip was varied as d, 0.5d and 0.25d, where d is the diameter of encased stone column. In the 3rd series of test loading was done on SC and ESC reinforced with horizontal strip of GI and PVC mesh, embedded in pond ash bed, with a variation in the placement and position of strip in the SC. Finite element analysis of 1st and 2nd series of test was done using software package PLAXIS and the values attained are compared with the experimental results.

4.2 PROPERTIES OF POND ASH AND STONE AGGREGATES

4.2.1 Index Properties

Specific gravity of pond ash and stone aggregates was found as per IS: 2720 (Part III) and obtained as 1.97 and 2.67 respectively. Specific gravity of pond ash is found to be lower than that of the conventional earth material. The specific gravity of the pond ash depend upon the source of coal, degree of pulverization and firing temperature. In addition to this the pond ash is subjected to mixing with other foreign matters in the ash pond which to some extent alters its specific gravity. Grinding of coal to higher fineness increases the specific gravity of pond ash due to breaking of cenosphere and carbon particles. The pond ash consists of grains mostly of fine sand to silt size. Based on the grain-size distribution, the coal ashes can be classified as sandy silt to silty sand. Coefficient of uniformity and coefficient of curvature for pond ash are 8.33 and 0.75 respectively.

4.2.2 Grain size distribution of pond ash

Grain size distributions pond ash was conducted as per IS: 2720 part (IV) for coarse fractions and hydrometer analysis was conducted for finer particles. Grain size distribution curve of pond ash is presented in Fig-3.1. Coefficient of uniformity (C_u) and coefficient of curvature (C_c) of pond ash are presented in Table-3.1.

4.2.3 Maximum and Minimum Density of Stone aggregates

Minimum and maximum dry density of stone aggregate was determined as per IS-2720 part (14). Minimum dry density was determined by filling the standard mold with aggregates to their loosest state. Maximum dry density was determined with respect to their densest state using vibrating table and putting a surcharged weight over it, as per provisions of IS-2720 part (14). The maximum and minimum dry densities of stone aggregate obtained are presented in Table-3.1.

4.2.4 OMC & MDD of pond ash

Light compaction test was carried out on pond ash as per IS: 2720 (Part VII). The OMC & MDD of pond ash is found out to be 25% & 1.17g/cc respectively. Heavy compaction test was conducted on pond ash and the OMC & MDD was obtained as 22% & 1.28g/cc. Plot between water content(%)-dry unit weight(kN/m^3) for light compaction test and heavy compaction test is plotted in Fig-3.2 and 3.3 respectively.

For preparation of pond ash bed compaction test was carried out on standard proctor mold.

Sample was placed in three equal layers and each layer was given 5 no of blows with hammer. Maximum dry density and optimum moisture content from the test was obtained as 1.15 g/cc and 28% respectively. This OMC and MDD were used for the preparation of pond ash bed. This OMC and MDD make the pond ash workable so that pipe can be inserted smoothly inside it for making stone column. Graph between water content (%)-dry unit weight (kN/m^3) for pond ash compacted in a standard proctor mold with 5 no of blows is plotted in fig-3.4.

4.2.5 Shear parameters of Pond ash

The shear parameters of pond ash sample compacted to their corresponding OMC and MDD were determined as per IS: 2720 (Part 13). These specimens were of size 60mm×60mm×25mm deep and sheared at a rate of 1.25 mm/minute. The shear strength parameters of the compacted specimens were determined from normal stress versus shear stress plots and it is given in fig-3.5. Cohesion and angle of internal friction was found to be 0.11kg/cm^2 and 34˚ respectively.

4.2.6 Shear parameters of stone aggregates

Direct shear test was conducted on stone aggregate compacted at 30% and 60% relative density. The test was conducted on a mold of size 150mm×150mm×75mm and the specimen were sheared at a rate of 1.25mm/minute. The shear strength parameters of the compacted specimens were determined from normal stress versus shear stress plots and it is given in fig-3.6 and 3.7. Angle of internal friction was found to be 44˚ and 47˚ respectively for stone compacted at 30% and 60% relative density.

4.3 Tensile strength of Reinforcing Material

Tensile strength test was carried out on reinforcing materials that is GI and PVC net. The properties obtained are given in Table.3.2. It is found that Young's modulus and load at peak achieved by GI sheet is 27.63 and 28.70 times that of PVC mesh, whereas strain at peak is more for PVC mesh than GI sheet and it is found to be 15.35 times higher than GI sheet.

4.4 LOAD DEFORMATION BEHAVIOR OF ENCASED STONE COLUMN (ESC)

4.4.1 Effect of Slenderness Ratio and Relative Density

The slenderness ratio of encased stone column (ESC) was varied as 0.5, 1, 2, 3 and 4. UCS test was carried out on this ESC by varying the relative density (30%, 60% or 90%) of the stone aggregates. Load corresponding to various deformations was noted and stress-strain curve is plotted.

4.4.1.1 Stress-strain variation for ESC (l/d=0.5, 1, 2, 3 and 4) compacted at different relative density

UCS test was carried out on encased stone column having slenderness ratio of 0.5,1,2,3 and 4 at a relative density of 30%, 60% and 90%. Load was applied at a strain rate of 1.25mm/min. The stress-strain curves are given below.

Fig-4.1 shows the curve between stress-strain of SC for different slenderness ratio, where the stone mass was compacted at 30% relative density. From the curve the maximum stress carried by SC having slenderness ratio of 0.5, 1, 2, 3 and 4 as 312.08kPa, 169.15kPa, 128.39kPa, 116.27kPa and 88.22kPa respectively. As the slenderness ratio decreases the load carrying capacity of SC increases. In the present test condition it is observed that there is a small variation in load carrying capacity of SC having slenderness ratio of 2, 3 and 4.

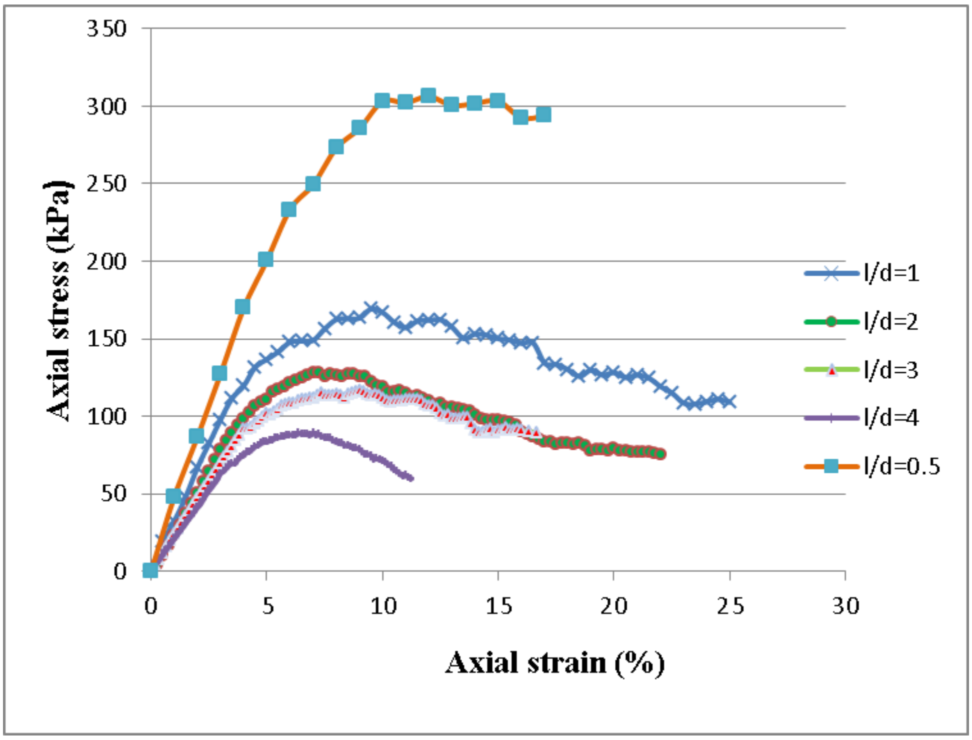


Fig-4.1 Stress-strain curve for SC (l/d=0.5, 1, 2, 3 and 4) at 30% relative density of aggregates

Fig-4.2 shows the stress-strain curve of SC at 60% relative density with the variation of slenderness ratio as 0.5, 1, 2, 3 and 4. There is an increasing trend in load carrying capacity with a decrease in slenderness ratio. SC consist of slenderness ratio of 0.5 carry much more load than the other slenderness ratio. It is calculated that maximum stress carried by SC with slenderness ratio of 0.5, 1, 2, 3 and 4 is 342.67kPa, 181.89kPa, 132.94kPa, 130.77kPa and 112.73kPa respectively.

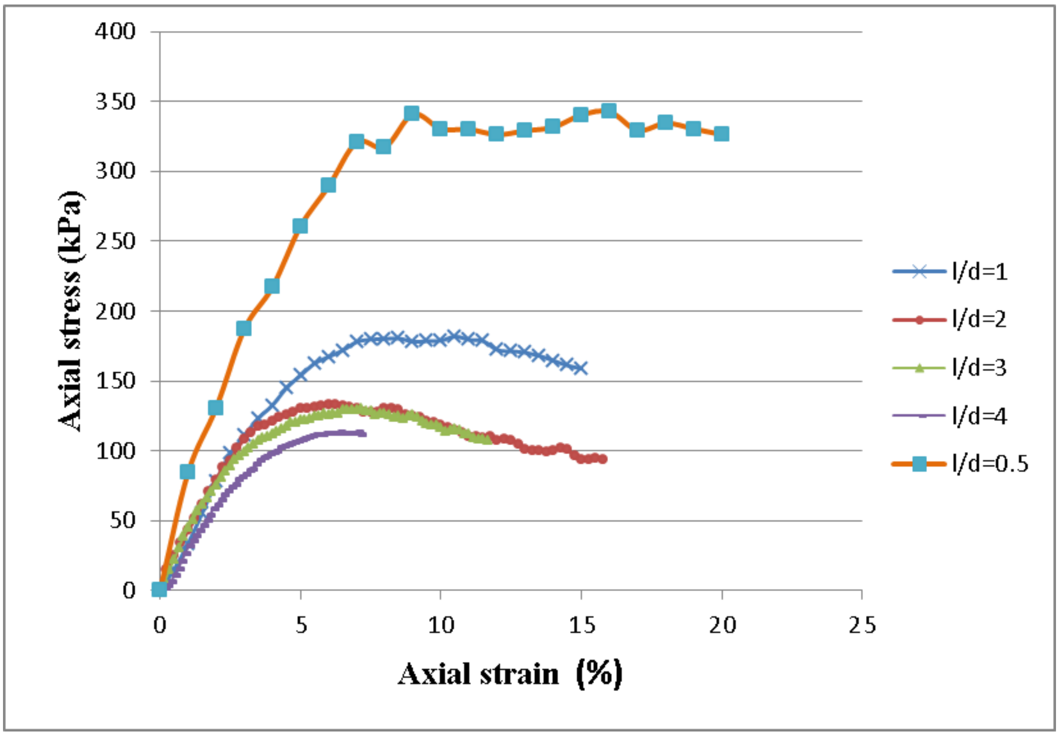


Fig-4.2 Stress-strain curve for SC (l/d=0.5, 1, 2, 3 and 4) at 60% relative density of aggregates

Fig-4.3 shows the stress-strain curve of SC at 90% relative density with the variation of slenderness ratio as 0.5, 1, 2, 3 and 4. With a higher degree of compaction and decrease in slenderness ratio, strength of SC increases by many folds. For the present test condition it was found that maximum stress carried by SC with slenderness ratio of 0.5, 1, 2, 3 and 4 is 450.96kPa, 188.94kPa, 136.05kPa, 132.35kPa and 125.64kPa respectively.

From the test conducted on SC by varying the slenderness ratio and relative density of stone mass it is observed that strength of SC is more for slenderness ratio of 0.5 and it has an increasing trend with the increase in relative density of aggregates. It is found that the increase in load carrying capacity of SC is less significant as the slenderness ratio increase from 2 to 4.

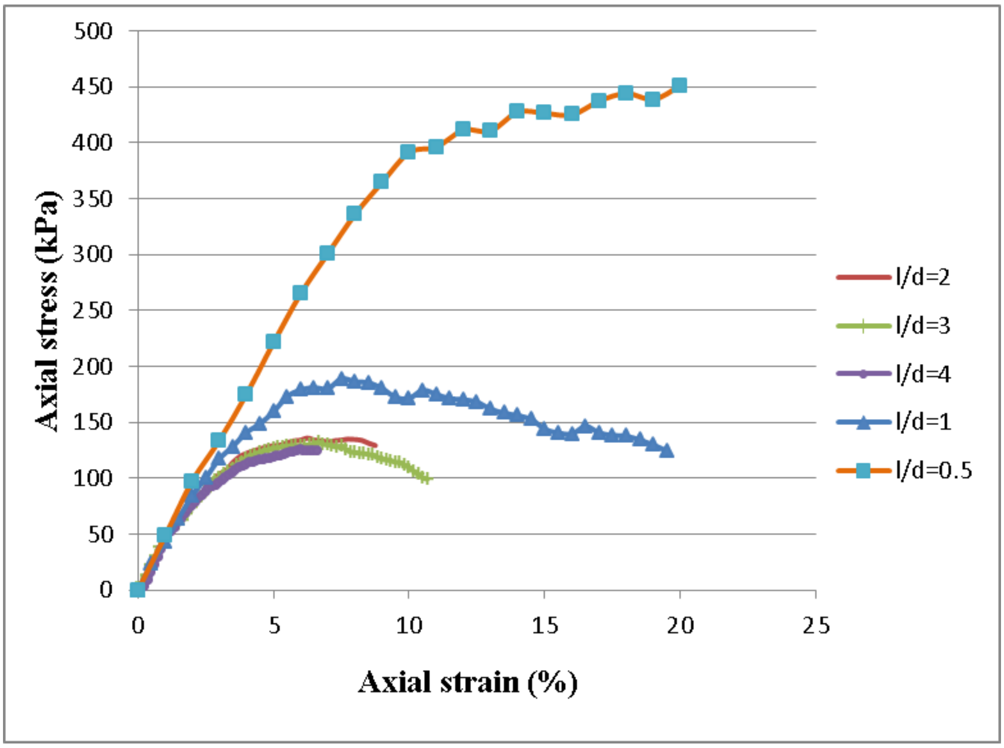


Fig-4.3 Stress-strain curve for SC (l/d=0.5, 1, 2, 3 and 4) at 90% relative density of aggregates

Table-4.1 Comparison of ultimate stress and strain of SC for different slenderness ratio and relative density of stone aggregates

	Relative density (%) →	30	60	90	30	60	90
		Ultimate stress (kPa)			Strain (%)		
Slenderness ratio of SC	0.5	312.08	342.67	450.96	19	16	20
	1	169.15	181.89	188.94	9.5	10.5	7.5
	2	128.39	132.94	136.05	7	6.25	6.25
	3	116.27	130.77	132.35	9	7.16	6.67
	4	89.22	112.73	125.64	7	7	6

Table 4.1 gives a comparative study on ultimate stress and strain of SC with an alteration in slenderness ratio and relative density of compacted stone aggregates. Secant modulus was determined at 7.5% axial strain from the stress strain graph plotted above. It is found out that

stiffness of SC is directly proportional to the relative density of stone mass and inversely proportional to slenderness ratio. The values of secant modulus obtained for different slenderness ratio and relative density is given in table-4.2

Table-4.2 Secant modulus at 7.5% axial strain for SC having different slenderness ratio and relative density

Relative density (%) →	30	60	90
Slenderness ratio (l/d) ↓	$(E_{sec})_{7.5\%}$ at 7.5% axial strain (kPa)		
0.5	3480	4267	4476
1	2086	2141	2519
2	1674	1741	1790
3	1506	1709	1714
4	1148	1503	1675

4.4.1.2 Modes of Failure of ESC

In the present test condition different modes of failure was observed for different slenderness ratio of ESC. ESC having slenderness ratio of 0.5 fails mainly due to the rupturing of encasement. Bulging is the cause for the failure of stone column with a slenderness ratio of 1 and 2, whereas ESC having slenderness ratio 3 and 4 fails due to buckling effect

4.4.2 Effect of horizontal strip reinforcement

In the present test condition circular strips of PVC net and GI sheet was taken as horizontal reinforcement. Slenderness ratio was taken as 2 and 3. The relative density was varied as 30%, 60%, and 90%. The spacing between the strips was varied as d, 0.5d, and 0.25d.

4.4.2.1 SC having slenderness ratio of 2 reinforced with circular strip of GI and PVC at various spacing

In the encased SC circular strip of GI and PVC was placed at d, 0.5d and 0.25d spacing, where d is the diameter of the SC. the no of circular strips required to achieve the spacing of d, 0.5d and 0.25d, are 1, 3 and 7 respectively. It is observed that as the spacing between the strips is reduced the load carrying capacity of the SC will increase. The maximum failure stress is obtained for higher relative density and closer spacing between the strips. Failure is mainly due to bulging action of SC. Bulging is less when there is a closer spacing between the strips.

Fig-4.4 Bulging of SC reinforced with GI strip at d spacing

Fig-4.5 SC reinforced with GI strip at 0.5d spacing

Fig-4.6 SC reinforced with PVC strip at 0.5d spacing

Fig-4.7 SC reinforced with PVC strip at 0.25d spacing

Fig-4.8 SC reinforced with GI strip at 0.25d spacing

Fig-4.4, 4.5, 4.6, 4.7 and 4.8 shows the bulging of SC reinforced with GI at d spacing, with GI strips at 0.5d spacing, PVC strips at 0.5d spacing, PVC strips and GI strips at 0.25d spacing respectively. Placement of horizontal GI strip arrests the bulging at the aggregate-reinforcement interface. Practically no bulging is found in the plane where the GI nets were provided but the PVC nets were unable to arrest the bulging. The bulging shape of SC reinforced with PVC strip is similar to that of ESC without horizontal reinforcement, which indicates that PVC strip are not effective in arresting the bulging. This may be due to lower stiffness value of PVC nets.

4.4.2.2 Stress-Strain Curve for SC (l/d=2) reinforced with GI and PVC strips

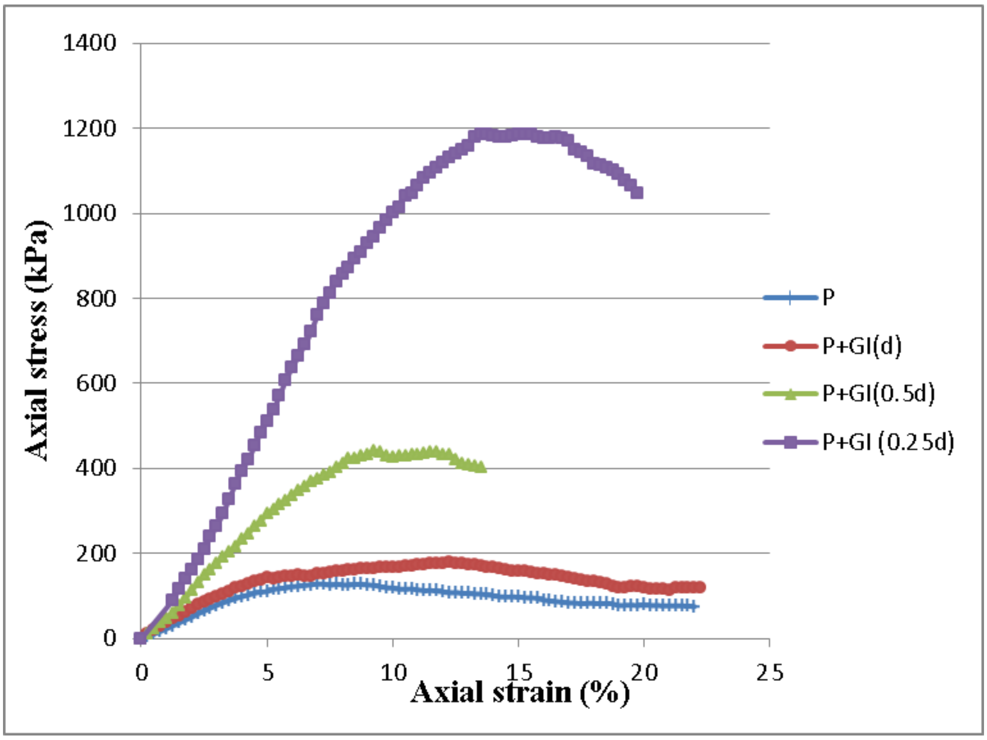


Fig-4.9 Stress- Strain curve for SC reinforced with GI at 30% Id

Fig-4.9 shows the plot between the stress-strain of an encased SC reinforced with GI circular strip at a relative density of 30%. It is observed that the maximum stress obtained for encased SC with no circular strip, SC with strip at a spacing of d, 0.5d, and 0.25d is 128.40kPa, 179.05kPa, 440.21kPa, and 1186.44kPa respectively. It shows that the maximum stress is increased by 1.39, 3.43, and 9.24 due to the insertion of GI strips at d, 0.5d, and 0.25d respectively. It is clearly visible that in the initial stage stress is increasing linearly with strain and after the failure stress it is decreasing. Due to the placement of GI strip at closer spacing the load carrying capacity of SC is increased by many folds, as it increases the stiffness of SC.

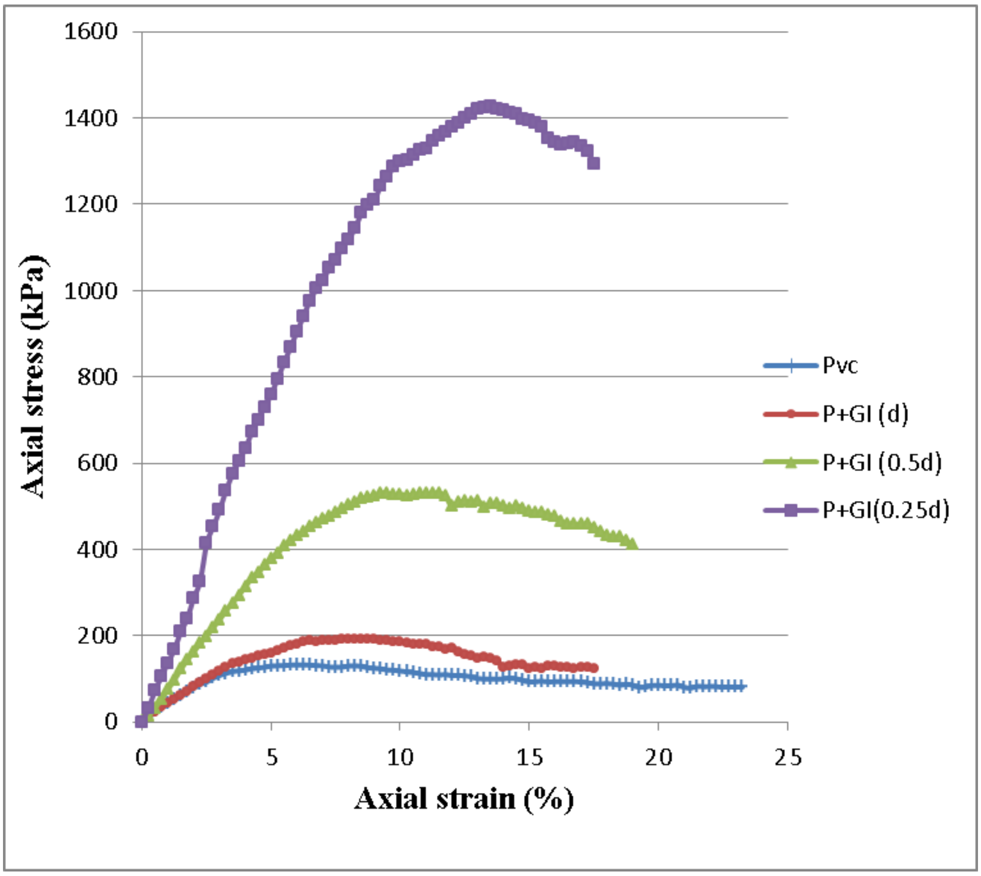


Fig-4.10 Stress- Strain curve for SC reinforced with GI at 60% Id

Due to the insertion of GI strip at closer spacing the strength of SC is increased by many folds. The load carrying capacity of SC without any circular strip is much less than the SC reinforced with GI strip. It is found from the fig-4.10 that the maximum stress for SC without any circular strip, SC with GI strip at a spacing of d, 0.5d, and 0.25d is 132.94kPa, 193.23kPa, 531.75kPa, and 1425.62kPa respectively. This shows that the strength is increased by 1.45, 4, 10.72 times due to the placing of GI strip than SC without any GI strip.

From the fig-4.11 it is observed that the strength of SC is increased remarkably with the insertion of GI sheet and closer spacing between the strips. It is found out that the maximum stress for SC without any circular strip, SC with strip at d, 0.5d, and 0.25d spacing is 135.19kPa, 206.70kPa, 649.56kPa, and 1546.45kPa respectively. This shows that the strength is increased by 1.53, 4.80, 11.44 times due to the placing of GI strip than SC without any GI strip.

From the fig-4.9, 4.10, 4.11 it is seen that relative density of stone mass and spacing of GI strip has great impact on load carrying capacity of SC. The strength of SC is higher for higher relative density and minimum spacing between the strips.

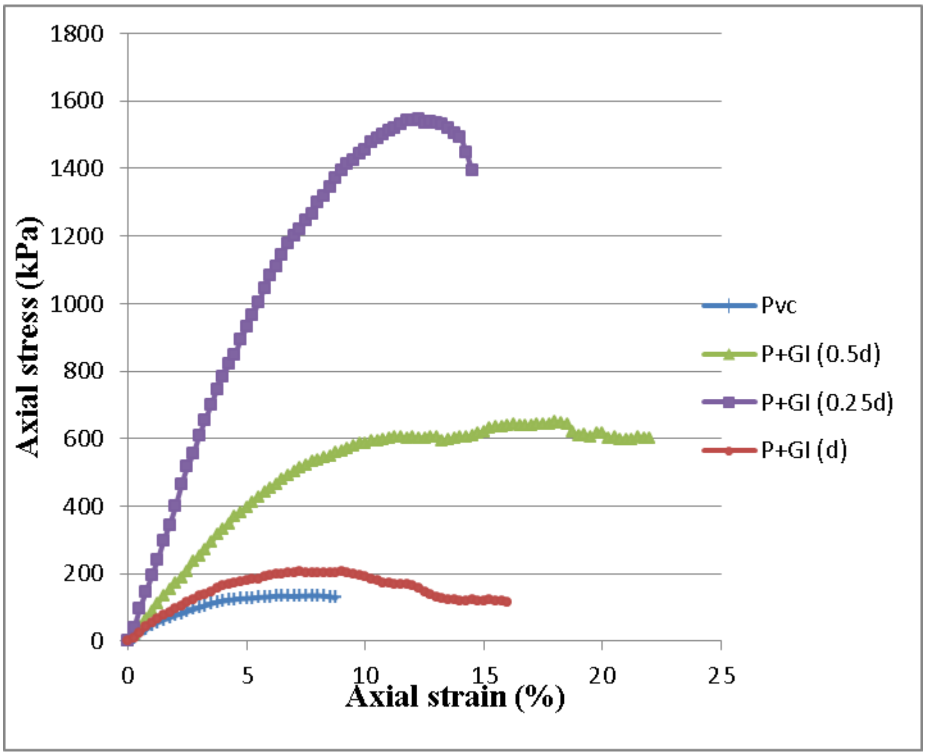


Fig-4.11 Stress- Strain curve for SC reinforced with GI at 90% Id

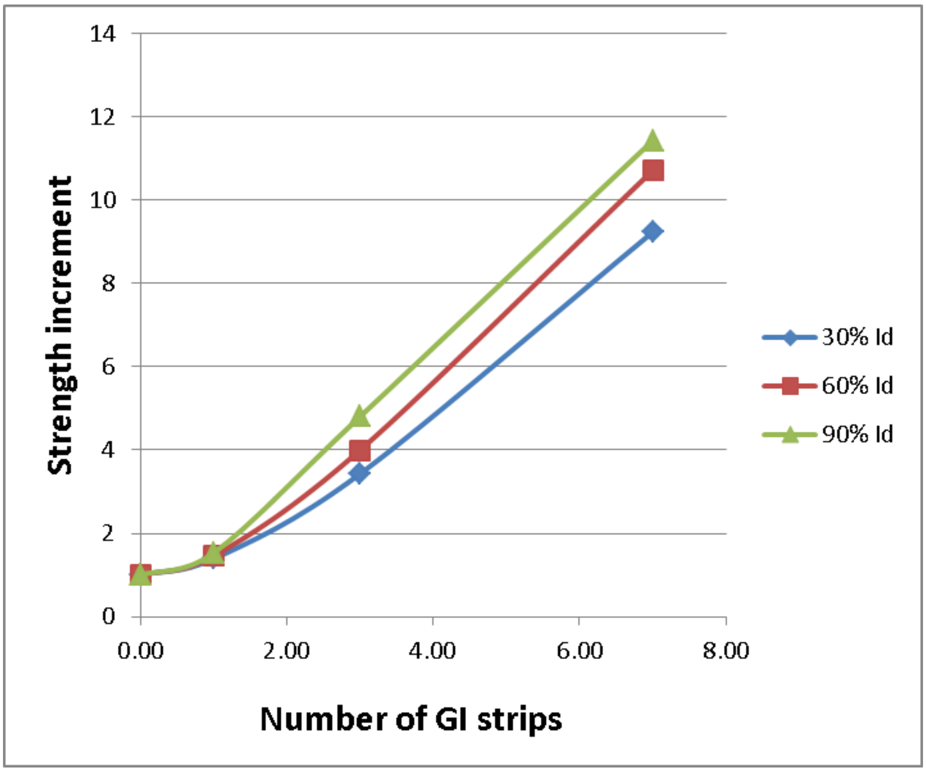


Fig-4.12 Number of GI strips-Strength increment for SC of l/d=2

Fig-4.12 shows the variation of strength increment with no. of GI strips. The more the no. of GI strips in a SC of 200mm height means the closer the spacing between the strips. It is noticed that the strength of SC is more for 90% relative density and it increases linearly with the no. of strips and relative density. Strength of SC is a function of relative density of stone mass, and spacing between the strips. The load carrying capacity of SC is directly proportional to relative density, no. of strips and inversely proportional to the spacing between the strips.

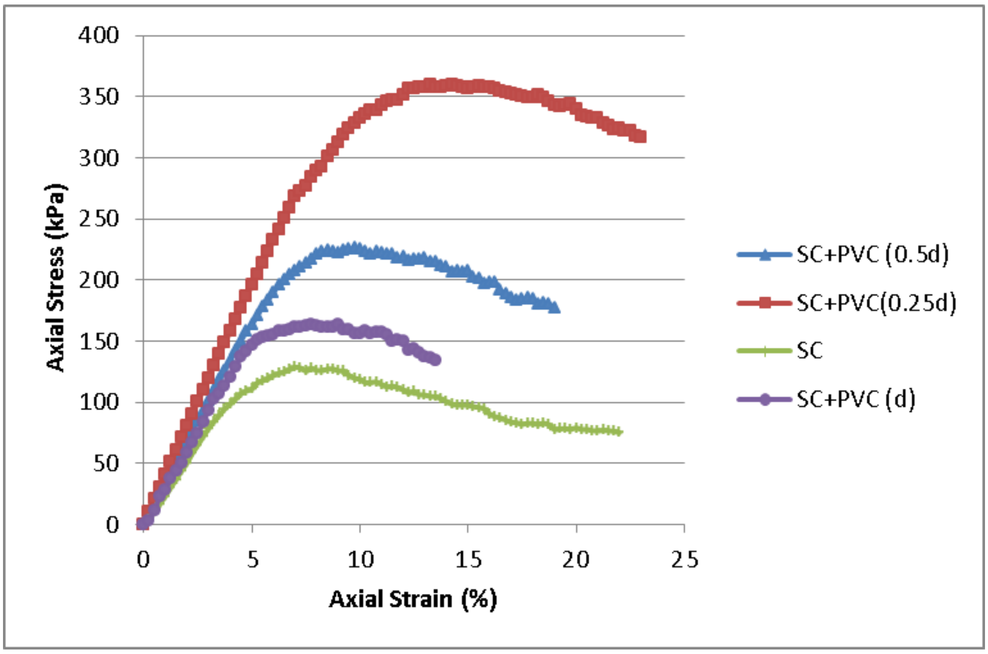


Fig-4.13 Stress- Strain curve for SC reinforced with PVC at 30% Id

Fig-4.13 shows the variation in stress-strain for SC of l/d=2 reinforced with circular PVC strips at 30% relative density. The maximum stress is increased as the spacing between the strips decreases. The maximum stress for SC without any circular strip, SC with PVC strip at d, 0.5d, and 0.25d spacing is calculated as 128.40kPa, 163.22kPa, 226.10kPa, and 360.06kPa respectively. This shows that the strength is increased by 1.27, 1.76, 2.80 times due to the placing of PVC strip than SC with no PVC strip.

Fig-4.14 indicates that the maximum stress for SC without any circular strip, SC with PVC strip at d, 0.5d, and 0.25d spacing is calculated as 132.94kPa, 179.81kPa, 233.11kPa, and 370.39kPa respectively. This indicates that the strength is increased by 1.35, 1.75, 2.79 times due to the placing of PVC strip than SC with no PVC strip. In the initial phase the stress has a linear relationship with strain. For the SC with no circular strip strength is less as compared to SC reinforced with circular strip at different spacing.

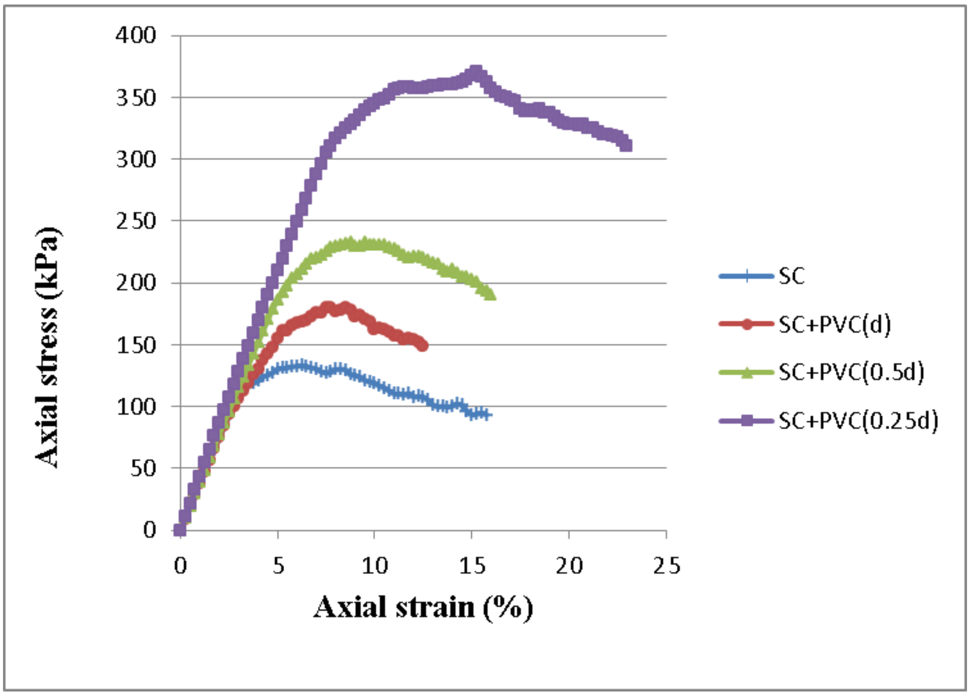


Fig-4.14 Stress- Strain curve for SC reinforced with PVC at 60% Id

Fig-4.15 shows that the maximum stress for SC without any circular strip, SC with PVC strip at d, 0.5d, and 0.25d spacing is calculated as 135.19kPa, 193.36kPa, 254.30kPa, and 419.87kPa respectively. This indicates that the strength is increased by 1.43, 1.88, 3.10 times due to the placing of PVC strip than SC with no PVC strip. In the initial phase the stress has a linear relationship with strain. For the SC with no circular strip the strain is higher and strength is less as compared to SC reinforced with circular strip at different spacing.

Placement of circular strip reinforcement increases the strength of SC. For the present test variable GI strip are found to be more effective than PVC strip. Further the improvement in load carrying capacity is more pronounced if the relative density of stone mass is higher. Circular GI strip placed at a spacing of 0.25d with relative density of stone mass 90% enhance the load carrying capacity by 11.44 times over the ESC.

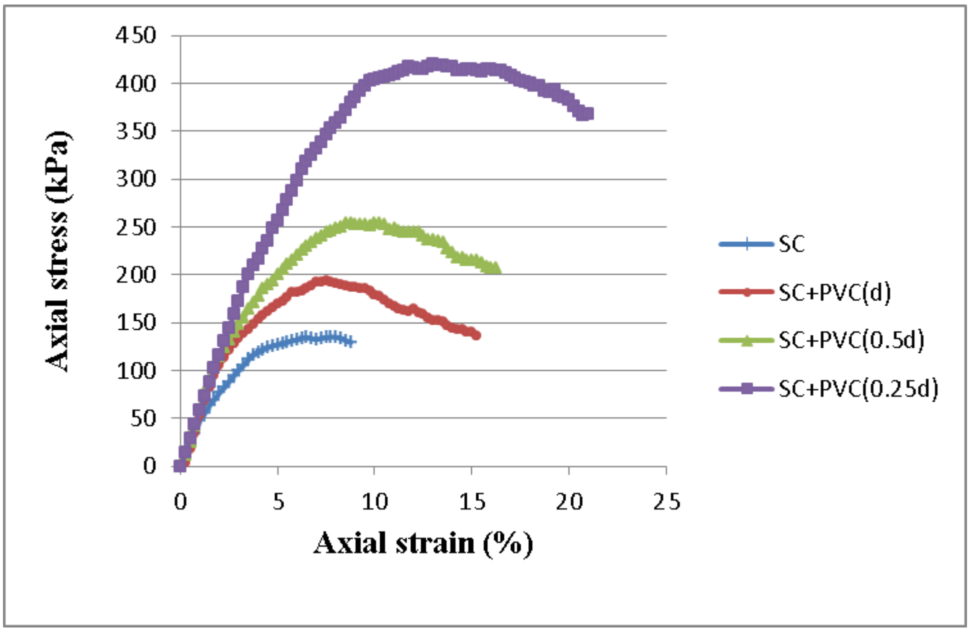


Fig-4.15 Stress- Strain curve for SC reinforced with PVC at 90% Id

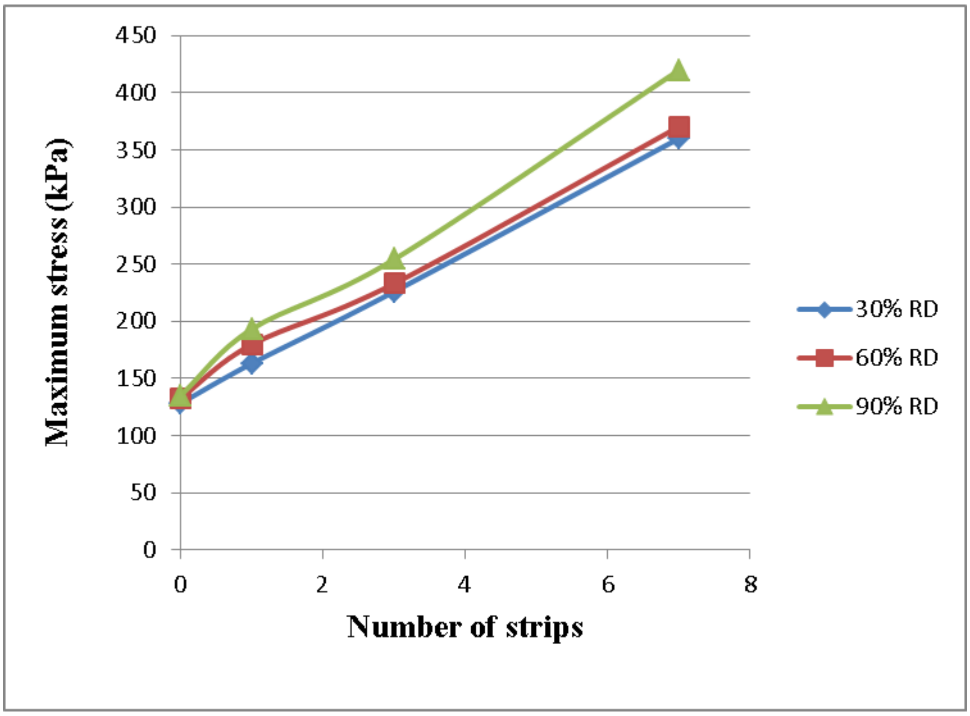


Fig-4.16 Number of Strips- Maximum stress of SC (l/d=2) reinforced with PVC strips

Fig-4.16 shows the variation of maximum stress with number of strips and relative density. It is found that stress increases with the increase in number of strips and relative density. For the present test sample relative density has less impact on enhancing the strength of SC. For greater spacing of strips at the three relative density (30%, 60%, and 90%) the increase in maximum stress is very less.

Table-4.3 Comparison of maximum stress of SC reinforced with PVC and GI strip with varying relative density of aggregates

NO OF STRIPS	PVC MAXIMUM STRESS (kPa)			GI MAXIMUM STRESS (kPa)		
	30% RD	60% RD	90% RD	30% RD	60% RD	90% RD
0	128.4	132.94	135.19	128.4	132.94	135.19
1	163.22	179.81	193.36	179.05	193.23	206.7
3	226.1	233.11	254.3	440.21	531.75	649.56
7	360.06	370.39	419.87	1186.44	1425.62	1546.45

A comparative study was done on the maximum stress obtained by SC reinforced with increasing no of PVC and GI strips with increase in relative density of aggregates. The results are given in Table-4.3. GI being a stiffer material than PVC, its placement in SC enhances the load carrying capacity by many folds. Closer packing of stone aggregates and increasing no of strips increase the stress.

4.4.3 SC having slenderness ratio of 3 reinforced with circular strip of GI and PVC at various spacing

Encased SC of 100mm diameter and 300mm length was taken. Circular strips of GI and PVC net were placed at a spacing of d, 0.5d and 0.25d. No of circular strips required to achieve this spacing of d, 0.5d and 0.25d are 2, 5 and 11 respectively. SC having higher relative density and lesser spacing between the strips has higher strength. In the present test condition failure of SC is due to bending as it has more slenderness ratio.

Fig-4.17 SC reinforced with GI strip at d spacing

Fig-4.18 SC reinforced with GI strip at 0.5d spacing

Fig-4.17 and 4.18 shows the ESC (l/d=3) reinforced with GI strip at d and 0.5d spacing respectively. Placement of horizontal GI strip arrests the bulging at the aggregate-reinforcement interface. Practically no bulging is found in the plane where the GI nets were provided.

4.4.3.1 Stress-Strain Curve for SC (l/d=3) reinforced with GI and PVC strips

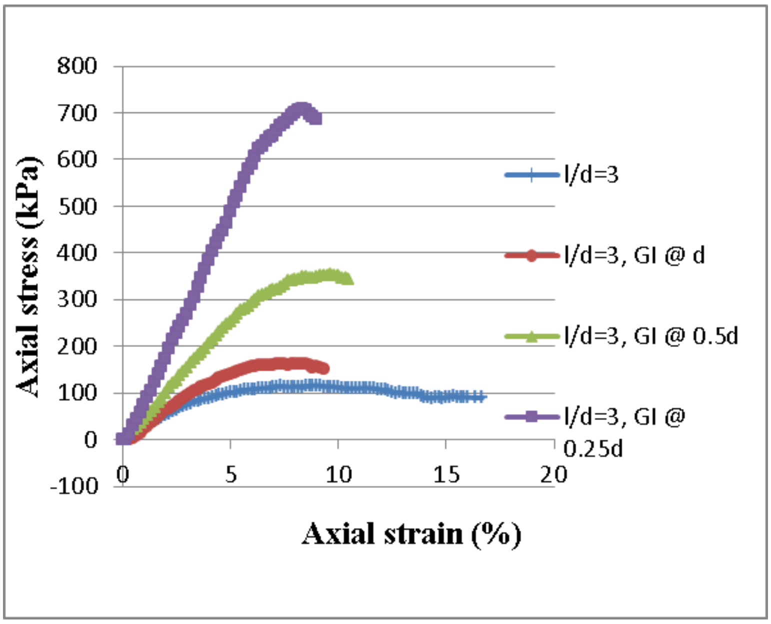


Fig-4.19 Stress-strain curve for SC reinforced with GI strip at 30% Id

Fig-4.19 shows the variation of stress versus strain for SC reinforced with GI strips for various configuration of spacing. It is calculated that the maximum stress for the SC without any strip is 116.27kPa. The maximum stress for SC reinforced with circular strip of GI at a spacing of d, 0.5d, and 0.25d is obtained as 163.02kPa, 353.77kPa, and 707.10kPa respectively. Because of the insertion of the GI strip at d, 0.5d, and 0.25d the strength is increased by 1.40, 3.04, and 6.08 times that of SC with no reinforcement. The use of GI strip inside the SC at minimum spacing makes the SC stiffer to withstand higher load.

Fig-4.20 indicates the stress-strain behavior of SC for different combination circular reinforcement of GI at 60% relative density. Stress increases linearly with strain at the initial stage of loading. In the case of SC having no reinforcement of GI strips it is observed that initially the stress increases linearly with strain, and in the later stage the stress is not increasing significantly with the increase in strain. The maximum stress for SC with no strip, strip at d, 0.5d, and 0.25d is found to be 130.77kPa, 180.14kPa, 504.36kPa, and 961.52kPa respectively. Introduction of GI strips at nearer spacing increase the strength of SC by many folds. It is calculated that due to the addition of the GI strip at d, 0.5d, and 0.25d the strength is increased by 1.38, 3.86, and 7.35 times that of SC with no reinforcement.

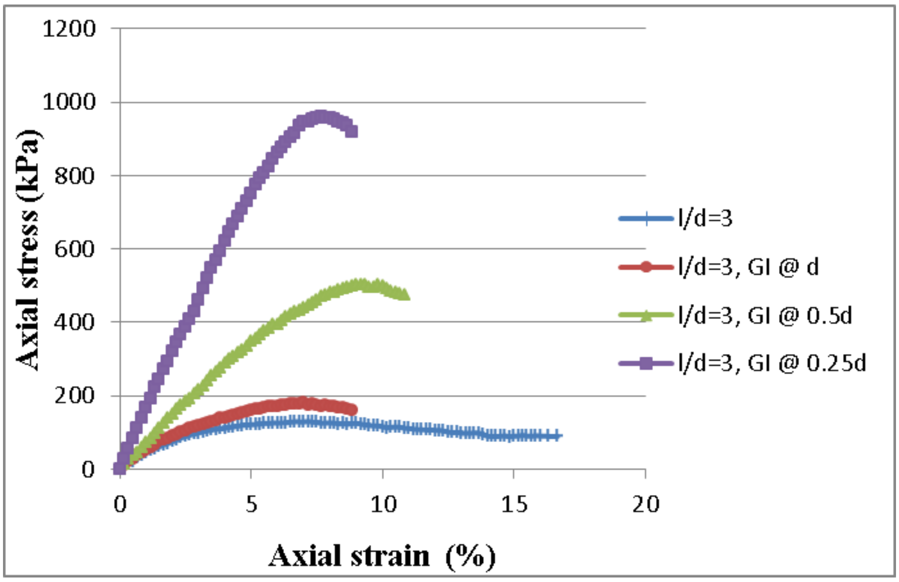


Fig-4.20 Stress-strain curve for SC reinforced with GI strip at 60% Id

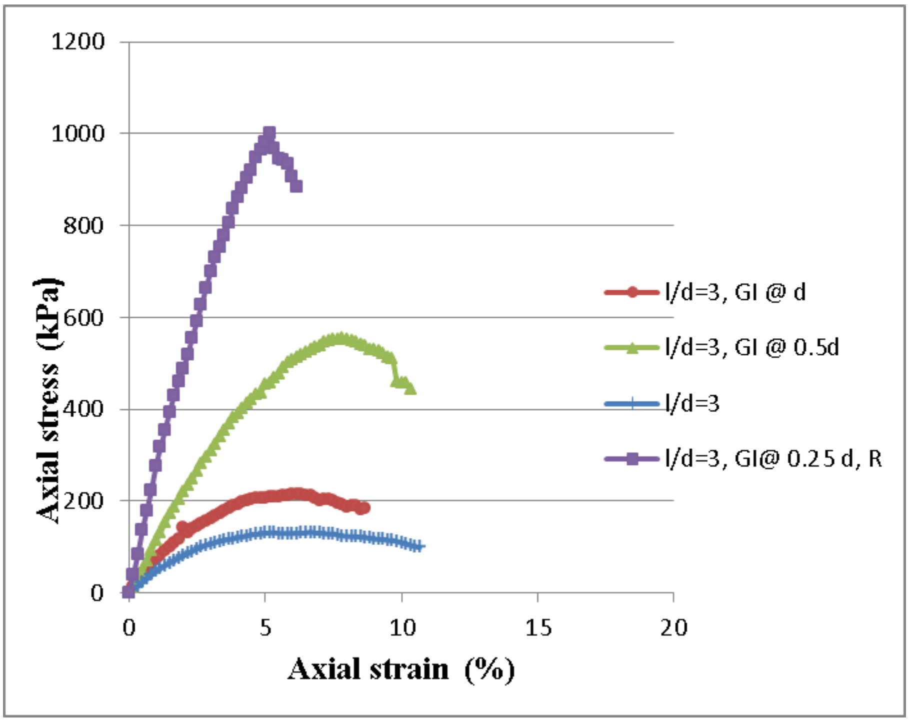


Fig-4.21 Stress-strain curve for SC reinforced with GI strip at 90% Id

From the fig-4.21 it is noticed that the strength of SC is increasing by several times because of the placing of GI strips at different arrangement of spacing. The maximum stress achieved by the SC with no strip, strip at d, 0.5d, and 0.25d is 132.35kPa, 214.59kPa, 552.98kPa, and 1001.62kPa respectively. This shows that the strength is increased by 1.62, 4.18, and 7.57 times for SC reinforced with GI strip at d, 0.5d, and 0.25d spacing.

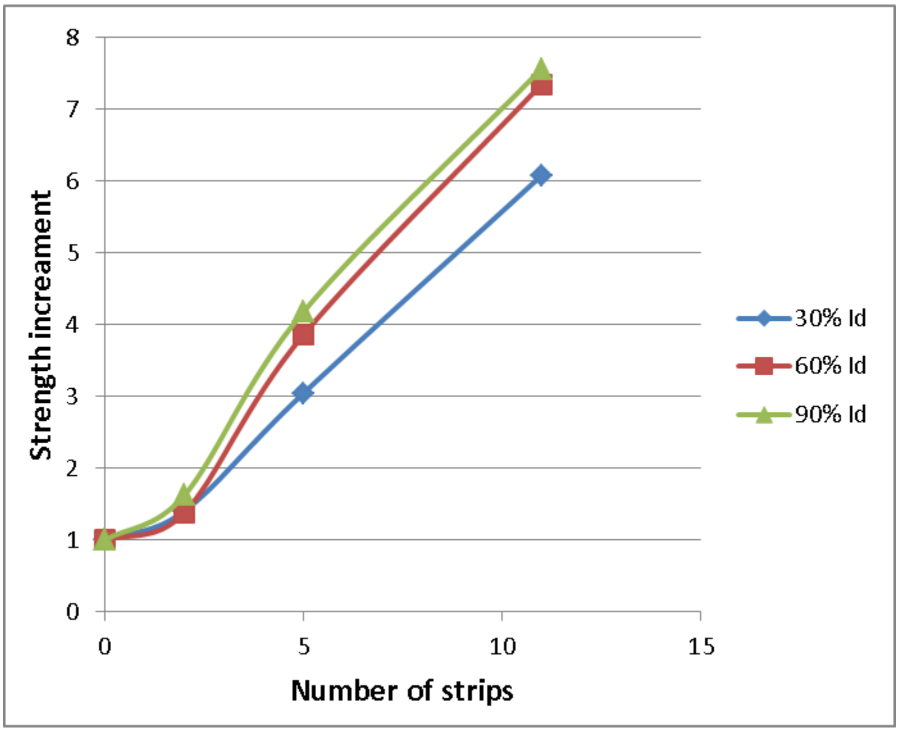


Fig-4.22 Number of strips-Strength increment for SC of l/d=3

Fig-4.22 shows the variation of strength increment with no. of strips used inside the SC and relative density of compacted stone mass. The strength is increasing linearly with the number of strips and relative density of aggregates. Closer packing of stone mass and minimum spacing of strips enhance the load carrying capacity of SC by many folds.

From fig-4.23 it is clearly visible that the strength of SC is amplified due to placing of PVC strips at nearer spacing. Maximum stress obtained for SC with no strip, SC with PVC strip at a spacing of d, 0.5d, and 0.25d spacing is 116.27kPa, 157.41kPa, 219.88kPa, and 313.52kPa respectively. This indicates a increment in strength by 1.35, 1.89, 2.70 times for SC with

PVC strips at d, 0.5d, 0.25d spacing to that of SC with no strips. The above curve illustrates that the stress is increasingly linearly with strain and after failure; stress is reducing non linearly with strain.

Fig-4.24 indicates the plot between stress-strain for SC strengthened with PVC strips at various spacing at 60% relative density. Introduction of PVC strips inside the SC enhances the strength by several times and it is found that nearer spacing of strips gives higher failure stress. From the graph it is found that the maximum stress obtained for SC with no strip, SC with PVC strip at d, 0.5d, and 0.25d spacing is 130.77kPa, 171.33kPa, 224.23kPa, and 319.62kPa respectively. This indicates a increment in strength by 1.31, 1.71, 2.44 times for SC with PVC strips at d, 0.5d, 0.25d spacing to that of SC with no strips.

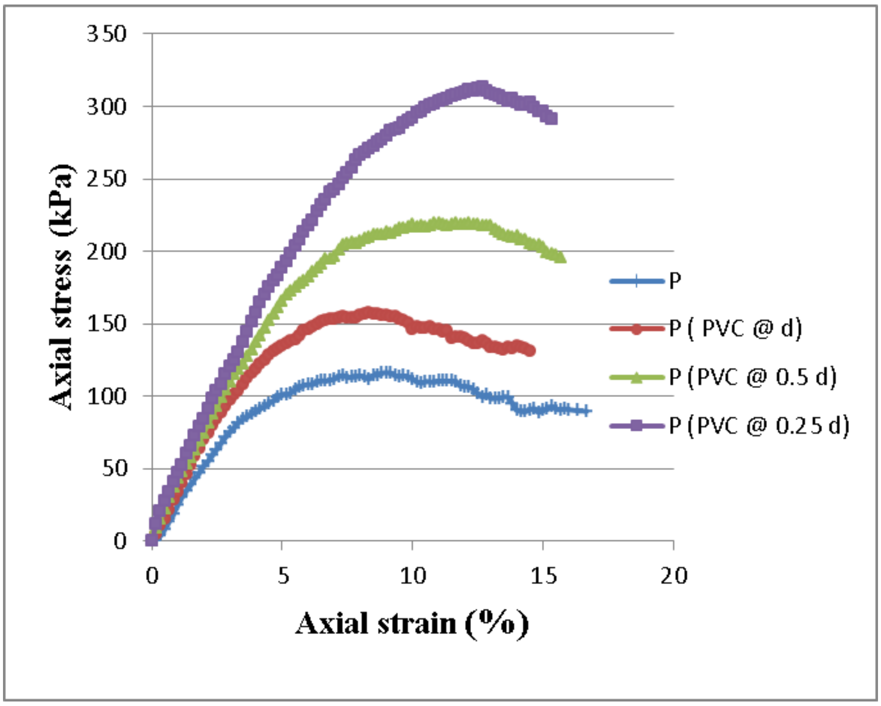


Fig-4.23 Stress-strain curve for SC (l/d=3) reinforced with PVC strip at 30% Id

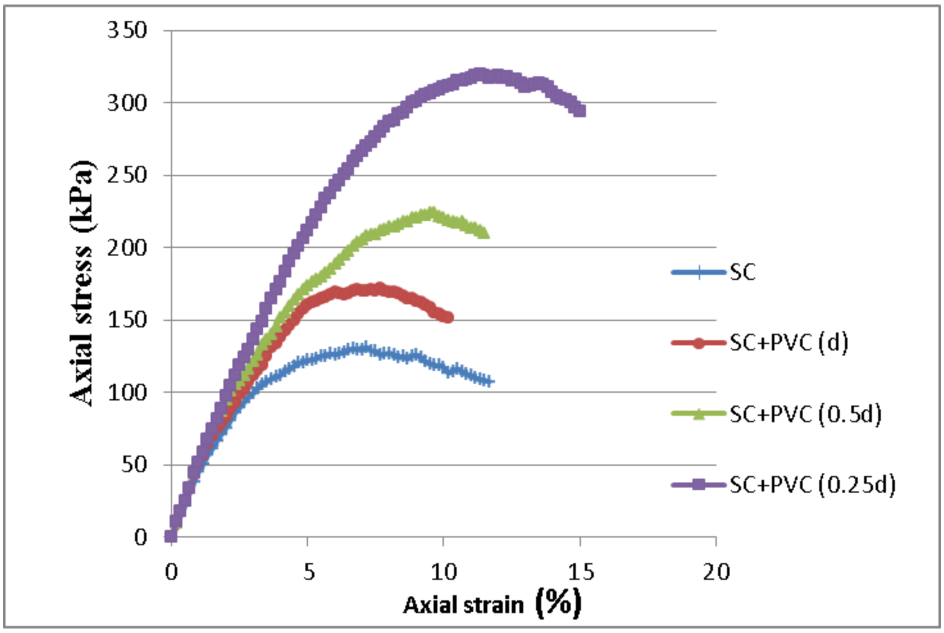


Fig-4.24 Stress-strain curve for SC (l/d=3) reinforced with PVC strip at 60% Id

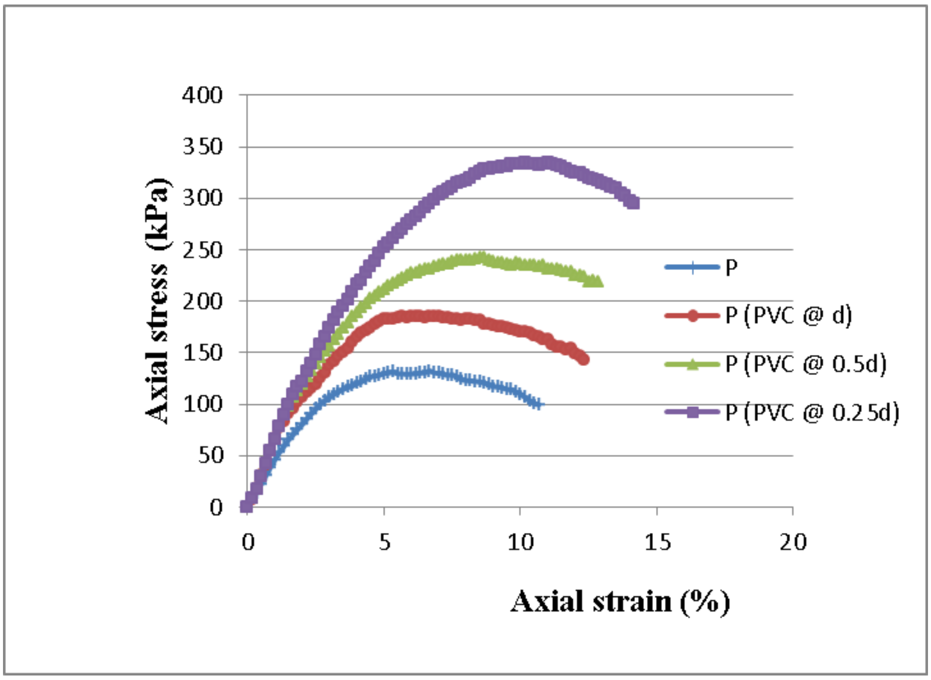


Fig-4.25 Stress-strain curve for SC (l/d=3) reinforced with PVC strip at 90% Id

From the fig-4.25 it can be observed that maximum stress achieved for SC with no strip, SC with PVC strip at d, 0.5d, and 0.25d spacing is 132.35kPa, 185.77kPa, 243.07kPa, and 334.48kPa respectively. This indicates a increment in strength by 1.40, 1.81, 2.53 times for SC with PVC strips at d, 0.5d, 0.25d spacing to that of SC with no strips.

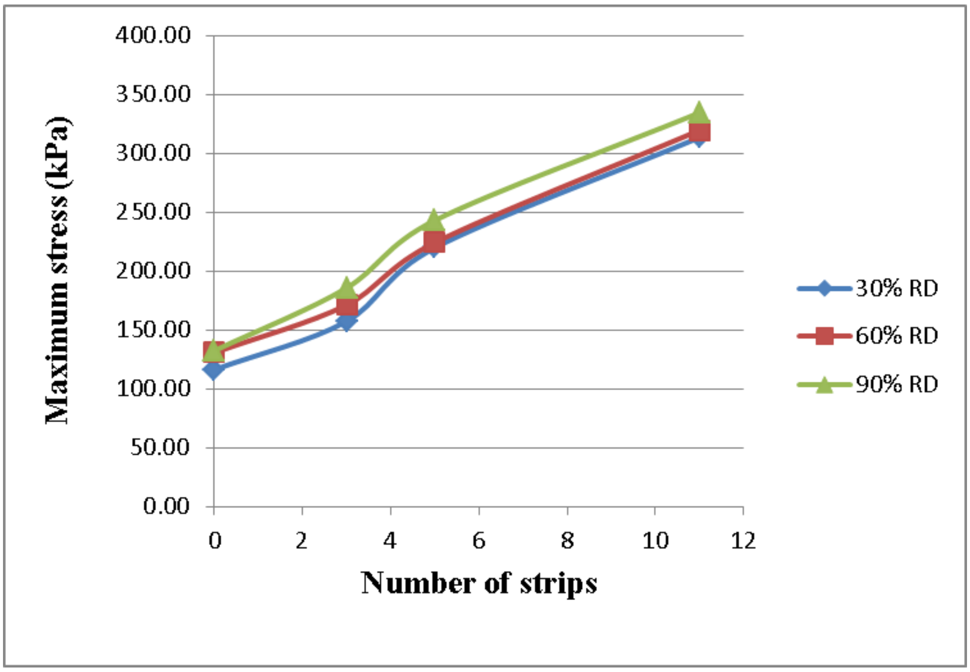


Fig-4.26 Number of strips-Maximum stress for SC (l/d=3) reinforced with PVC

From fig-4.26 it is perceived that the strength of SC is improved due to the insertion of more number of strips inside the column and higher relative density of compacted stone mass. There is a linear increment in stress with number of strips.Maximum stress is achieved for SC having higher relative density and closer spacing of strips. In the present test condition it is observed that the impact of relative density on strength is less significant. The improvement in strength due to change in relative density is very less. For higher relative density and reduced spacing of PVC strips the SC acts as a stiffer column and carries more load

A comparative study was done on SC having slenderness ratio of 3 reinforced with 0, 2, 5 and 11 numbers of PVC and GI strips and it is given in table-4.4. It is observed that insertion of GI strips makes the SC stiffer to carry high stress as compared to SC reinforced with PVC strips. The increase in load carrying capacity of SC is a function of stiffness of strips used, relative density of stone aggregates and number of strips. In the present test condition it is found that strength of SC is directly proportional to the no of strips, relative density of stone mass and stiffness of circular strips used.

Table-4.4 Comparison of maximum stress of SC (l/d=3) reinforced with PVC and GI strip with varying relative density of aggregates

NO OF STRIPS	PVC			GI		
	MAXIMUM STRESS (kPa)			MAXIMUM STRESS (kPa)		
	30% RD	60% RD	90% RD	30% RD	60% RD	90% RD
0	116.27	130.77	132.35	116.27	130.77	132.35
2	157.41	171.33	185.77	163.02	180.14	214.59
5	219.88	224.23	243.07	353.77	504.36	552.98
11	313.52	319.62	334.48	707.1	961.52	1001.62

.A comparative study was done ultimate stress and strain of SC having slenderness ratio of 2 and 3, reinforced with GI strip and PVC strip at d, 0.5d and 0.25d spacing and presented in table 4.5, 4.6

Table-4.5 Comparison of ultimate stress and strain of SC (l/d=2) reinforced with PVC and GI Strip

Type of horizontal strip	Relative density (%)	30	60	90	30	60	90
		Ultimate stress (kPa)			Strain (%)		
PVC	No strip	128.40	132.94	135.19	6.75	6.25	7.75
	At d spacing	163.22	179.81	193.36	7.75	8.5	7.5
	At 0.5d spacing	226.10	233.11	254.3	9.75	9.5	8.5
	At 0.25d spacing	360.06	370.39	419.7	13.25	15.25	13
GI	No strip	128.40	132.94	135.19	7	6.25	7.75
	At d spacing	179.05	193.23	206.70	7.75	8.25	7.25
	At 0.5d spacing	440.21	531.75	649.56	9.25	9	18
	At 0.25d spacing	1186.44	1425.62	1546.45	13.75	13.75	12.25

Table-4.6 Comparison of ultimate stress and strain of SC (l/d=3) reinforced with PVC and GI Strip

Type of horizontal strip	Relative density (%)	30	60	90	30	60	90
		Ultimate stress (kPa)			Strain (%)		
PVC	No strip	116.27	130.77	132.35	9	7.17	6.67
	At d spacing	157.41	171.33	185.77	8.33	7.67	6.17
	At 0.5d spacing	219.88	224.23	243.07	10.83	9.67	8.67
	At 0.25d spacing	313.52	319.62	334.48	12.67	11.33	10.16
GI	No strip	116.27	130.77	132.35	9	7.16	6.67
	At d spacing	163.02	180.14	214.59	8.33	7	6.16
	At 0.5d spacing	353.77	504.36	552.98	9.5	9.33	7.5
	At 0.25d spacing	707.1	961.52	1001.62	8.33	7.67	5.16

Secant modulus obtained from the stress strain curve defines the stiffness of SC. For the present test condition Secant modulus was obtained at 7.5% and 5% axial strain for SC having slenderness ratio of 2 and 3 respectively. The values thus obtained are given in table-4.7 and 4.8. It shows that stiffness of SC increases with relative density of compacted stone aggregates and closer spacing between the circular strip. GI being stiffer material than PVC makes the SC rigid to transmit additional load.

Table-4.7 Secant modulus at 7.5% axial strain for SC having slenderness ratio of 2 and relative density of 30%, 60% and 90%

Type of horizontal strip (hs)	GI strip			PVC strip		
Relative density (90%)	30	60	90	30	60	90
Specification of SC	$(E_{sec})_{7.5\%}$ at 7.5% axial strain (kPa)			$(E_{sec})_{7.5\%}$ at 7.5% axial strain (kPa)		
No hs	1674	1697	1790	1674	1697	1790
hs at d spacing	2078	2511	2726	2163	2395	2578
hs at 0.5d spacing	5214	6477	6971	2848	3003	3254
hs at 0.25d spacing	10502	14025	16603	3694	4065	4627

Table-4.8 Secant modulus at 5% axial strain for SC having slenderness ratio of 3 and relative density of 30%, 60% and 90%

Type of horizontal strip (hs)	GI strip			PVC strip		
Relative density (90%)	30	60	90	30	60	90
Specification of SC	$(E_{sec})_{5\%}$ at 5% axial strain (kPa)			$(E_{sec})_{5\%}$ at 5% axial strain (kPa)		
No hs	2027	2436	2596	2027	2436	2596
hs at d spacing	2792	3216	4144	2691	3201	3649
hs at 0.5d spacing	5045	6962	9076	3309	3479	4221
hs at 0.25d spacing	9752	15051	19617	3777	4228	5045

4.5 LOAD SETTLEMENT BEHAVIOR OF SC EMBEDDED IN POND ASH (PA) BED

SC of diameter 50mm and length 250 mm was embedded in compacted pond ash bed. Footing of diameter 50mm was taken and footing load test was conducted on the sample. In the present test condition ordinary SC, SC reinforced with GI strip and PVC mesh at various spacing, encased SC reinforced with GI strip at various spacing were considered.

4.5.1 Load-settlement behavior of pond ash and pond ash reinforced with SC

Footing load test was carried out on compacted pond ash bed and pond ash reinforced with SC of diameter 50mm and height 250mm. the load-settlement curve was obtained and the bulging behavior of SC was studied.

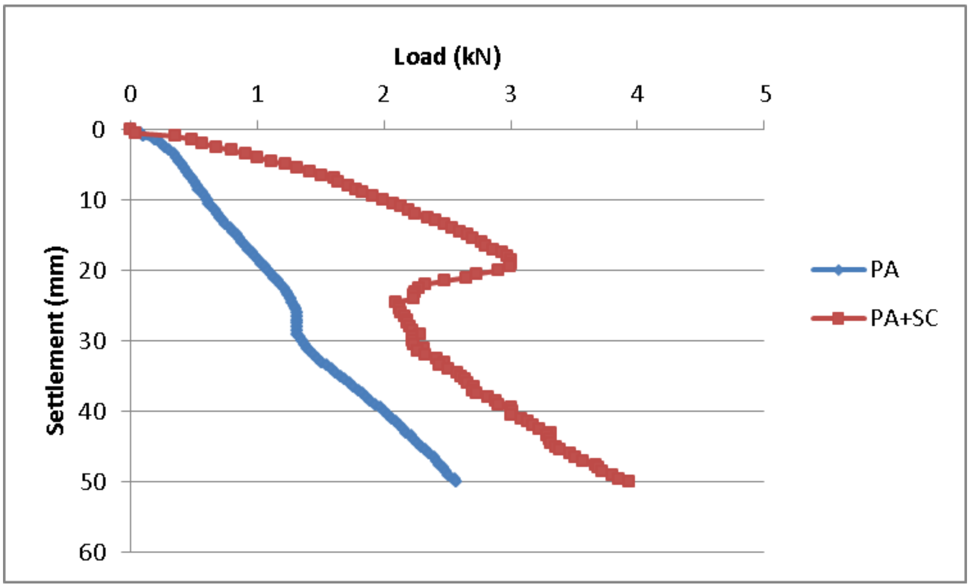


Fig-4.27 Load-settlement behavior of pond ash and pond ash reinforced with SC

It is observed from the fig-4.27 that pond ash has very low bearing capacity and it can be improved by placing SC. The graph shows that pond ash reinforced with SC failed and again starts increasing up to a higher value. Load taken by pond ash reinforced with SC at 50mm settlement is 1.53 times that of pond ash.

Fig-4.28 shows the deformed shape of SC after loading. It was observed that SC fails due to bulging and bulging occurs at D to 2D distance, where D is the diameter of the SC. Maximum bulging is found at a depth of 1.06d and its diameter at failure 1.28d for un encased SC

Fig-4.28 Bulging of SC

4.5.2 Load-settlement behavior of SC reinforced with circular PVC strips at various spacing, inserted in compacted pond ash bed

Footing load test was conducted on pond ash bed, pond ash reinforced with SC and pond ash with reinforced SC. The SC was reinforced with PVC mesh at various positions. PVC mesh was placed at D, 2D, 3D and 4D, where D is the diameter of SC. It was observed that SC fails due to bulging and bulging occurs at D to 2D distance. So an attempt was made to see the effect of placement of PVC mesh at 0.5D, 2 number of PVC mesh at 0.5D,D, 3 number of PVC mesh at 0.5D,D,1.5D, 4 number of PVC mesh at 0.5D,D,1.5D, 2D and 4 number of PVC mesh at D,2D,3D,4D. Load versus settlement curve is plotted for these cases. A comparative study was done to analyze the improvement in load carrying capacity of pond ash bed reinforced with SC and PVC mesh at different position and spacing.

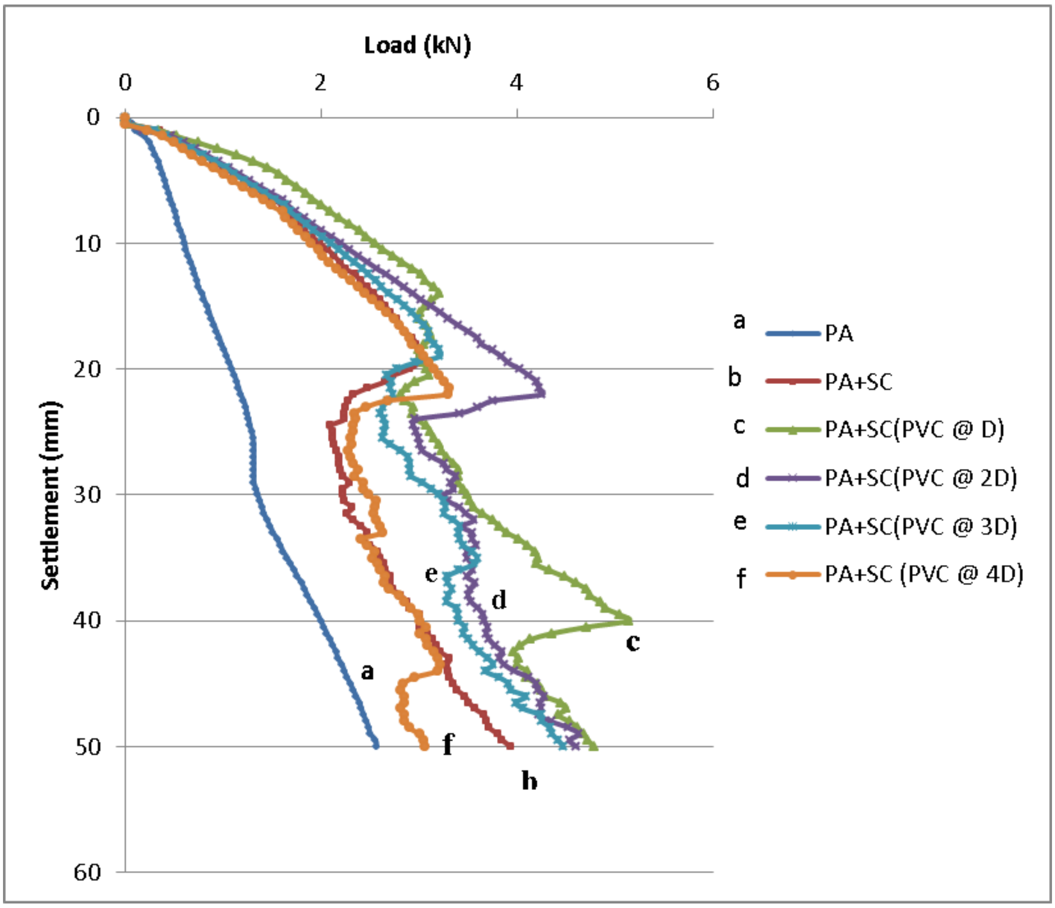


Fig-4.29 Load-settlement behavior of pond ash, pond ash reinforced with SC and SC reinforced with PVC mesh at various spacing

Fig-4.29 shows the load settlement behavior of pond ash bed reinforced with SC and SC prepared with PVC mesh at different position. It indicates that placement of PVC mesh increases the load carrying capacity of ordinary SC. Insertion of PVC mesh at large distance from the effective bulging zone has less impact on improving the load carrying capacity of SC.

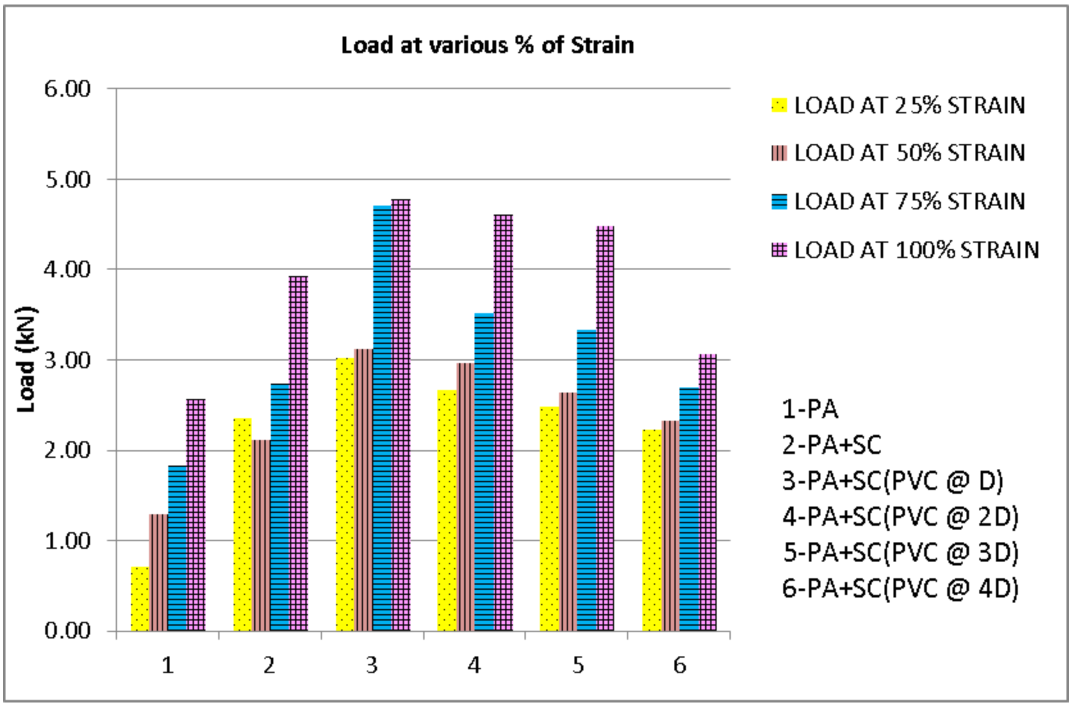


Fig-4.30 Variation of load at 25%, 50%, 75% and 100% of Strain with position of PVC strip in the SC

Fig-4.30 shows the variation of load carrying capacity of pond ash reinforced with SC and SC prepared with PVC mesh placed at various positions. It is clearly observed that the load carrying capacity increases with increasing % of strain and decreases with the increase in distance of PVC mesh. Further the load capacity is increased due to the insertion of PVC mesh at the effective bulging zone. Load carrying capacity of pond ash bed reinforced with SC was obtained at 50mm settlement and it is found out that the load carrying capacity is increased by 1.53, 1.86, 1.79, 1.74 and 1.19 times that of pond ash due to the placing of SC, SC with PVC mesh at D, SC with PVC mesh at 2D, SC with PVC mesh at 3D and SC with PVC mesh at 4D respectively.

Fig-4.31 shows the variation in load carrying capacity of pond ash, pond ash reinforced with SC and pond ash with SC where PVC strips were placed at various configurations. It was observed that SC fails mainly due to bulging and bulging occurs at D to 2D distance from the top of the SC. so the SC is again reinforced with a single PVC strip at 0.5D, 2 number of strips at 0.5D, D, 3 number of strips at 0.5D, D and1.5D, 4 number of strips at 0.5D, 1D, 1.5D and 2D and 4 number of strips at D, 2D, 3D and 4D. It is observed that there is a increment strength of pond ash bed reinforced with SC. The load carrying capacity is again enhanced by placing more number of strips at closer spacing near the bulging zone. From the present test it is observed that SC reinforced with 4 numbers of PVC strip at 0.5D, 1D, 1.5D and 2D results in high load carrying capacity of pond ash bed. Load carrying capacity of pond ash bed reinforced with SC was obtained at 50mm settlement and it is found out that the load carrying capacity is increased by 1.53, 1.60, 1.64, 1.82, 2.16 and 1.98 times that of pond ash due to the placing of SC, SC with PVC mesh at 0.5D, SC with PVC mesh at 0.5D and D, SC with PVC mesh at 0.5D,D and 1.5D, SC with PVC mesh at 0.5D, D, 1.5D and 2D SC with PVC mesh at D, 2D, 3D and 4D respectively.

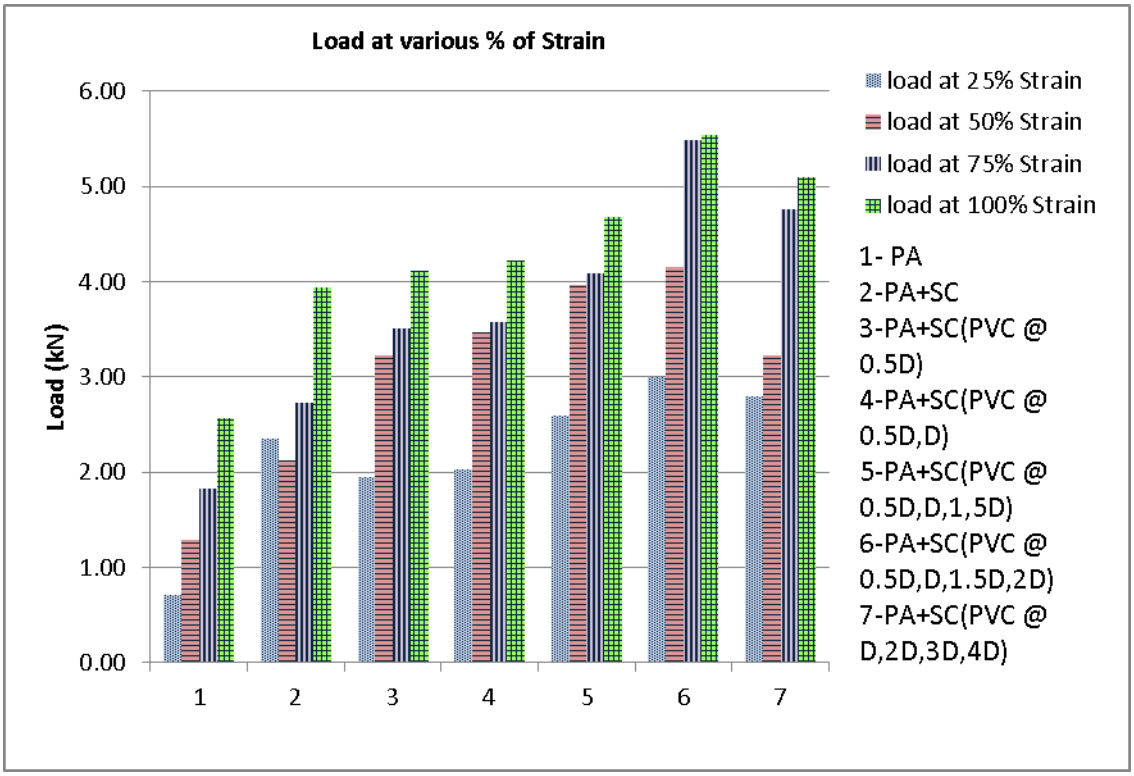


Fig-4.31 Load variation with strain and number of PVC strip and its position in the SC

4.5.3 Load-settlement behavior of ESC reinforced with number of GI strip placed at various position, embedded in compacted pond ash bed

The SC was encased with PVC mesh and it was embedded in compacted pond ash bed. Circular GI strip of diameter 50mm was placed at various positions. The diameter of the SC was taken as 50 mm. Different arrangements that were taken to place the GI strip in the ESC are as follows. Single GI strip was placed at 0.5D, 2 numbers of GI strip were placed at 0.5D and D, 3 numbers of strips were placed at 0.5D,D and 1.5D, 4 numbers of strips were placed at 0.5D,D,1.5D and 2D and 4 numbers of strips were placed at D, 2D,3D and 4D. Load versus settlement curve was plotted for various arrangement of the SC.

Fig-4.32 shows the load settlement behavior of pond ash, pond ash reinforced with SC, pond ash strengthen with encased stone column and pond ash with ESC where ESC were equipped with numbers of GI strips at various position. Pond ash being more compressible carries less load and its load carrying capacity is enhanced by many folds due to the placement of encased stone column. Further it is observed that load carrying is increased by a remarkable amount due to the placement of GI strips. It is perceived from the test that strength is higher for the SC where GI strips were placed at 0.5D, D, 1.5D and 2D.

Fig-4.33 shows the variation of load with 25%, 50%, 75% and 100% strain for pond ash, pond ash strengthen with SC, ESC and ESC with number of GI strips. Load carrying capacity increases with % increase in strain. ESC made up of GI strips at 0.5D, D, 1.5D and 2D results in high load carrying capacity of the resulting SC embedded in compacted pond ash bed. Load corresponding to 50mm settlement was obtained for different combinations of pond ash and ESC with GI strips. It is noticed that load carrying capacity is increased by 1.53, 1.71, 1.89, 2.55, 3.13, 3.37 and 2.25 times that of pond ash bed alone due to the placing of ordinary SC, ESC, ESC with single GI strip placed at 0.5D, ESC prepared with GI strips at 0.5D and

D, 3 numbers of strips placed at 0.5D,D and 1.5D, 4 numbers of strips placed at 0.5D,D,1.5D and 2D and 4 numbers of strips placed at D, 2D,3D and 4D respectively. Encasing the SC makes the SC stiffer to withstand higher load and load is increased by many folds when the ESC is prepared with GI strips at closer spacing near the bulging zone.

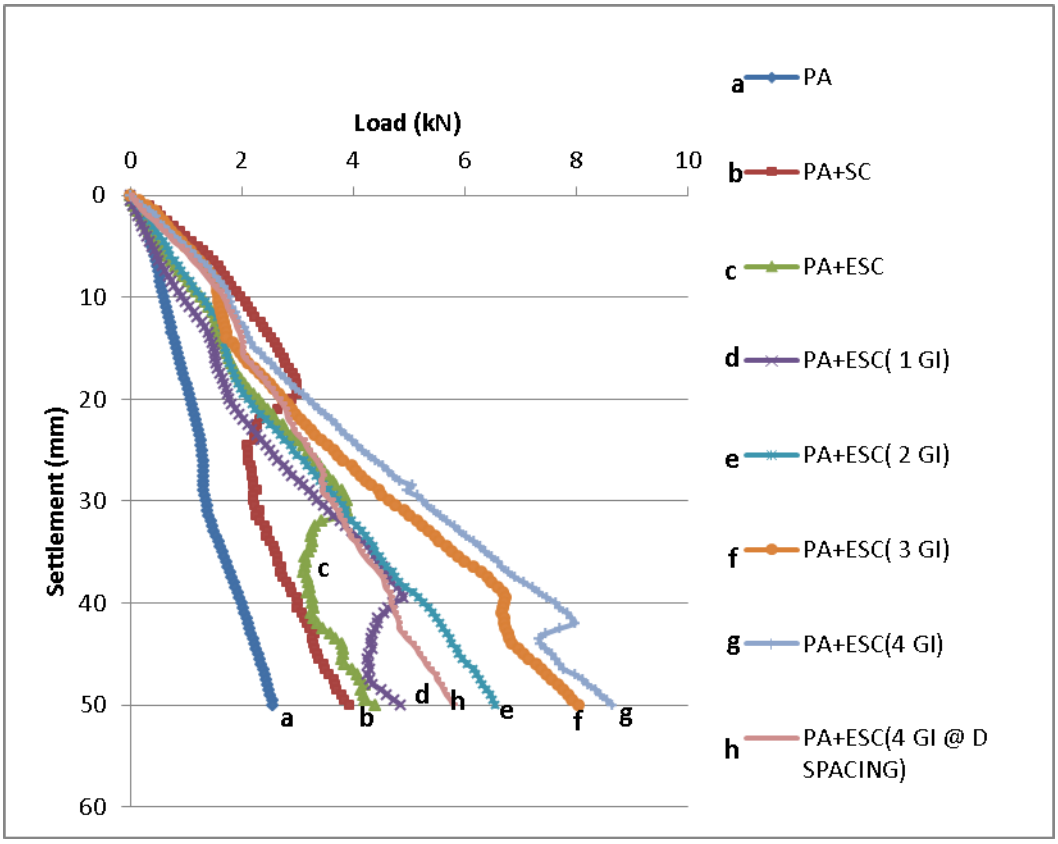


Fig-4.32 Load –settlement behavior of ESC prepared with various arrangement of GI strips.

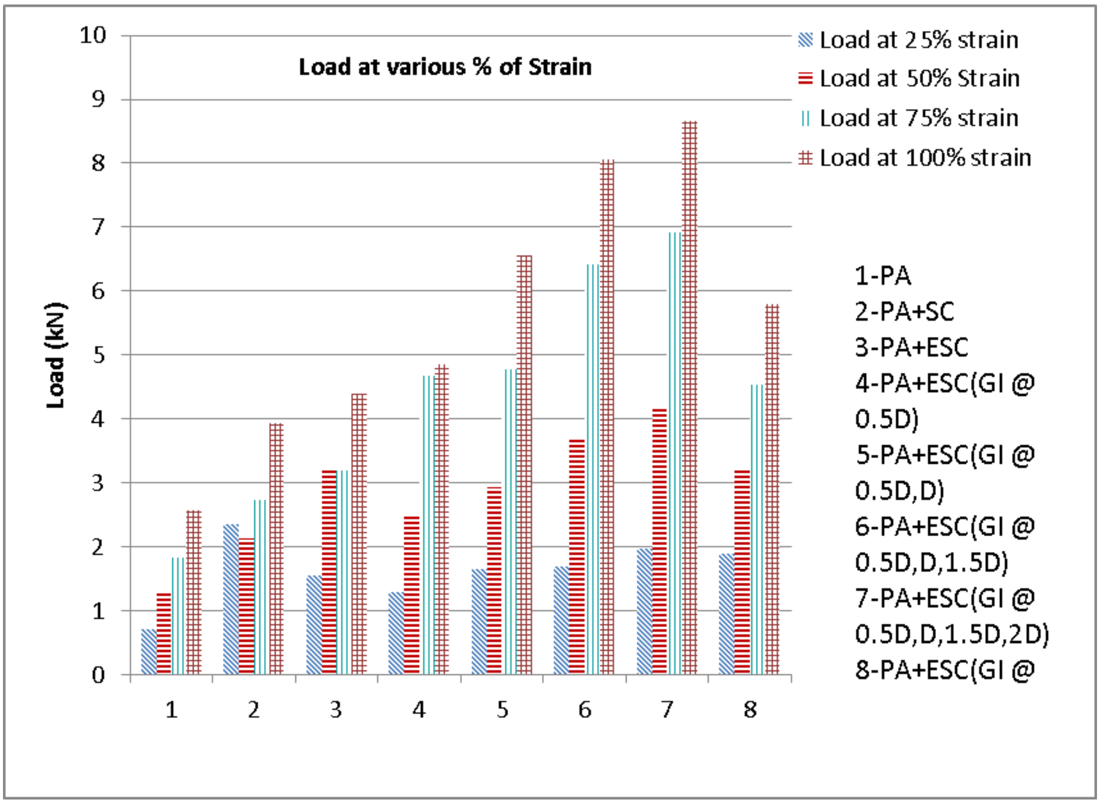


Fig-4.33 Variation of load with increment in % strain

4.5.4 Load-settlement behavior of SC reinforced with GI strips at various spacing, introduced in compacted pond ash bed.

Footing load test was conducted on ordinary SC and SC reinforced with GI strips at different positions, which is embedded in compacted pond ash bed. Pond ash possesses more compressibility and low bearing capacity. The load carrying capacity of the pond ash can be improved by inserting SC and it can be further improved by many folds due to insertion of GI strips at a closer spacing near the bulging zone.

Fig-4.34 shows that load carrying capacity of the pond ash bed alone is very less and it is improved by placing SC inside it. It is observed that the load carrying capacity is further enhanced by placing GI strips. The strength of SC introduced in pond ash bed is more where the SC is equipped with GI strips at closer spacing near the bulging zone. From the load-settlement curve it is observed that at 50mm settlement the load carrying capacity is increased by 1.53, 1.80, 2.19, 3.11, 3.33 and 1.94 times that of pond ash bed alone due to the placing of ordinary SC, SC with single GI strip placed at 0.5D, SC prepared with GI strips at 0.5D and D, 3 numbers of strips placed at 0.5D,D and 1.5D, 4 numbers of strips placed at 0.5D,D,1.5D and 2D and 4 numbers of strips placed at D, 2D,3D and 4D respectively.

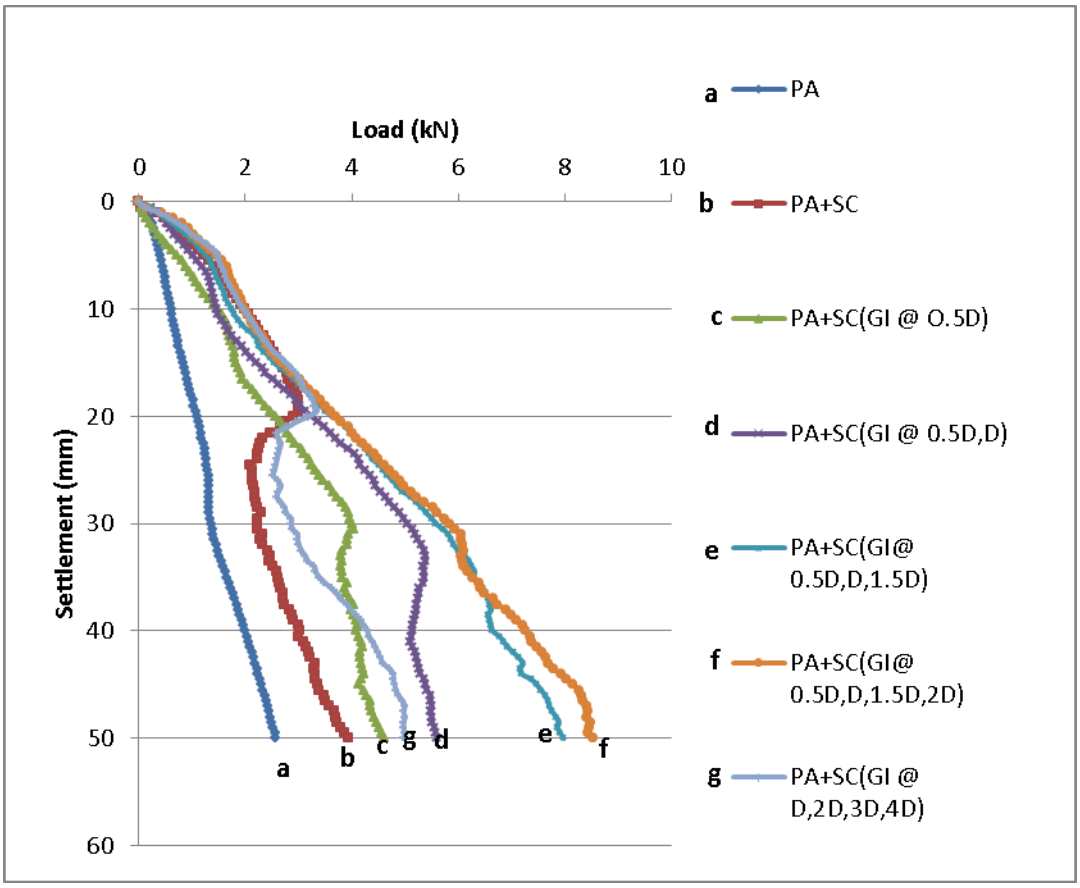


Fig-4.34 Load –settlement behavior of SC prepared with various arrangement of GI strips

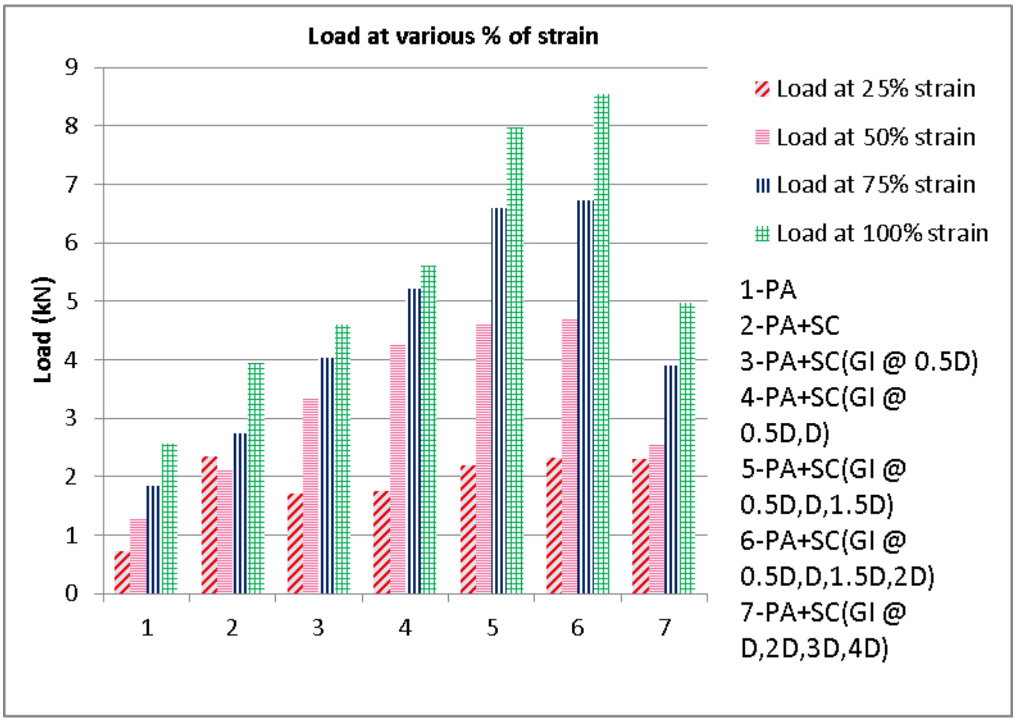


Fig-4.35 Variation of load with strain (%)

Fig-4.35 shows the variation of load carrying capacity of pond ash alone, pond ash reinforced with SC, pond ash with SC having GI strips at different position, at 25%, 50%, 75% and 100% strain. There is a gradual increase in load carrying capacity of SC placed in compacted pond ash bed with the increase in strain and number of GI strips. SC reinforced with 4 numbers of GI strips at 0.5D, D, 1.5D and 2D gives more strength to the column.

4.5.5 Comparative study on the behavior of SC and ESC reinforced with circular strips, embedded in compacted pond ash bed

A comparative study was carried out on the behavior of SC, ESC reinforced with GI strips and SC strengthen with PVC mesh placed at 0.5D, D, 1.5D and 2D. The SC was embedded in compacted pond ash bed. The increment in load carrying capacity of pond ash at 50mm settlement due the placement of SC and ESC reinforced with GI strip or PVC mesh was found out. The results obtained are as follows:

Fig-4.36 shows the improvement in load carrying capacity due to the insertion of SC, SC reinforced with single GI or PVC strip and ESC reinforced with GI strip at 0.5D with respect to the pond ash alone. It is found out that ESC with a single GI strip makes the SC stiffer to carry high load. The load carrying capacity is increased by 1.54, 1.61, 1.81 and 1.91 times than that of pond ash alone due to the placement of SC, SC having a PVC mesh, GI strip and ESC having a GI strip respectively.

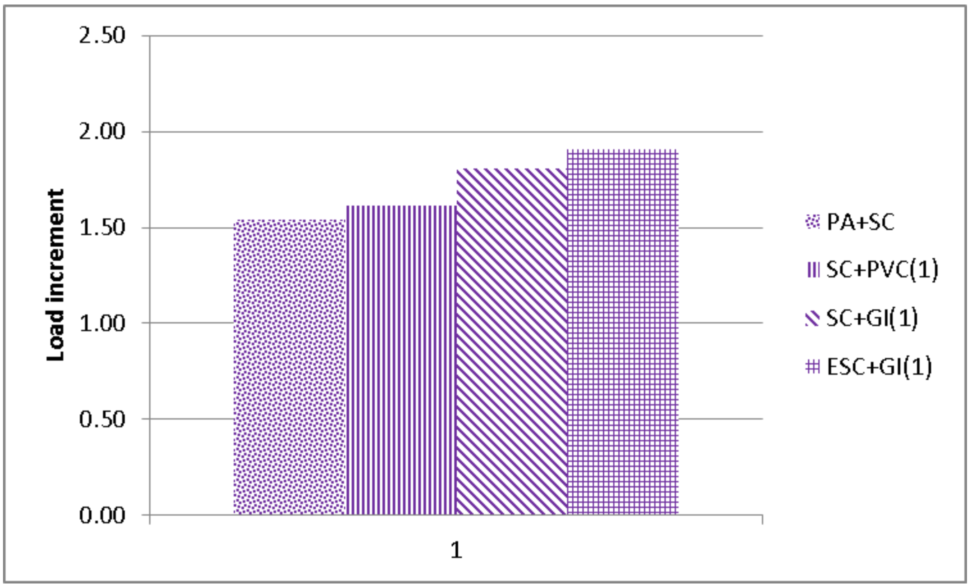


Fig-4.36 Load increment due to the placement of single circular strip placed at 0.5D in the SC

From the fig-4.37 it is obtained that the load carrying capacity of the pond ash bed is increased by 1.54, 1.66, 2.20 and 2.57 times because of the introduction of SC, SC reinforced with PVC mesh and GI strip at 0.5D and D, ESC reinforced with two GI strip at 0.5D and D in the pond ash bed.

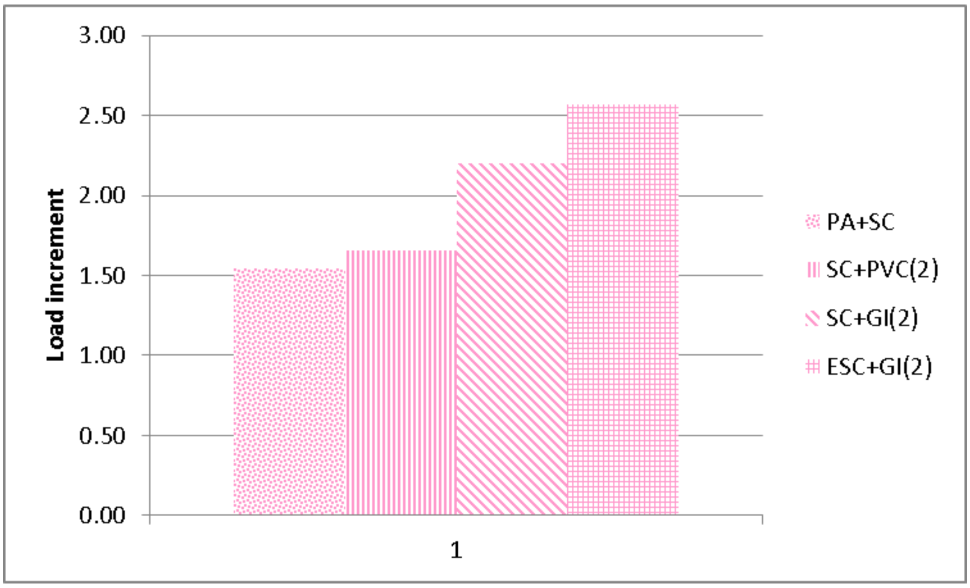


Fig-4.37 Load increment due to the placement of two circular strips placed at 0.5D and D in the SC

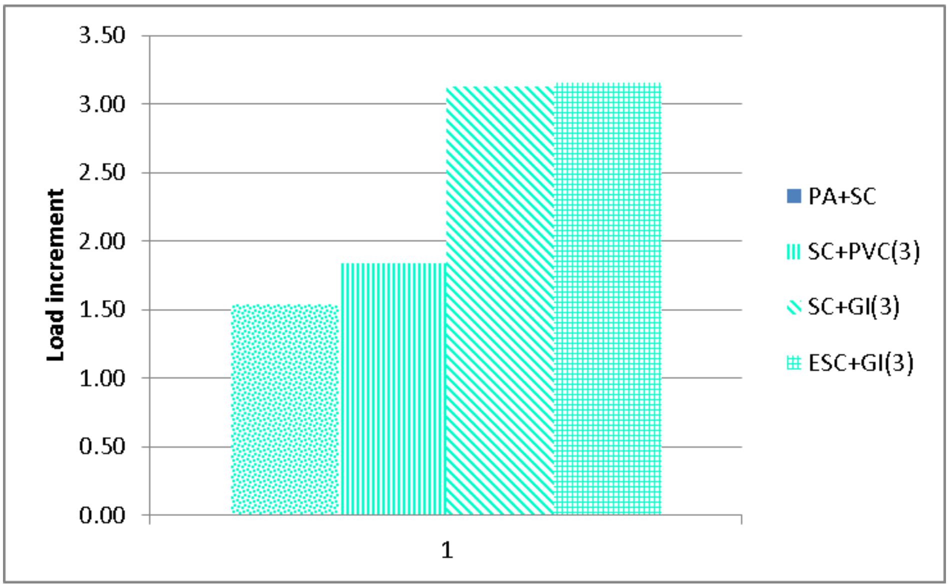


Fig-4.38 Load increment due to the placement of three circular strips placed at 0.5D, D and 1.5D in the SC

Placing of three circular strips in the SC makes it stiffer to carry a high load. The strips were placed at 0.5D, D and 1.5D in the SC. ESC with the above mentioned combination of strips carries more loads as compared to others. There is an increment in load by 1.54, 1.84, 3.12 and 3.16 times due to the placement of SC, SC with three PVC, GI strips and ESC with three GI strips that of the pond ash bed alone subjected to footing load test.

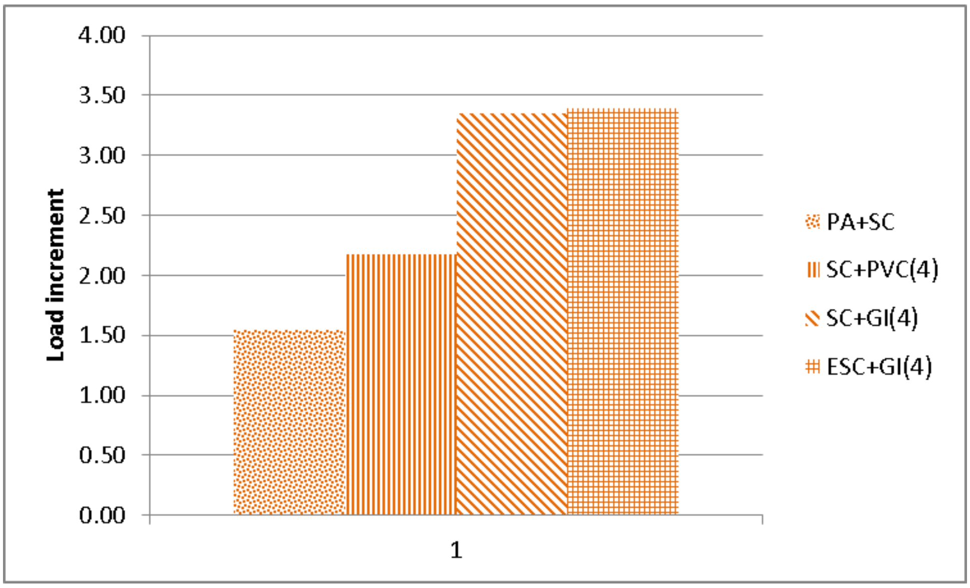


Fig-4.39 Load increment due to the placement of four circular strips placed at 0.5D, D 1.5D and 2D in the SC

Fig-4.39 shows the relative improvement in load carrying capacity of pond as bed reinforced with SC, SC reinforced with four PVC mesh, GI strip and ESC reinforced with GI strip at 0.5D, D, 1.5D and 2D. It is found out that at 50mm settlement the load carrying capacity is enhanced by 1.54, 2.17, 3.35 and 3.39 times due to the addition of SC, SC reinforced with four PVC mesh, GI strip and ESC reinforced with GI strip respectively.

From the fig-4.36, 37, 38 and 39 it can be observed that load carrying capacity of pond ash bed is improved by many folds due to the introduction of ESC with four numbers of GI strips placed at 0.5D, D, 1.5D and 2D. The above combination of GI strips makes the ESC stiffer to with stand a great load.

Full encasement of SC reduces the bulging diameter and it carries more load than ordinary

SC, embedded in pond ash bed. Further ESC reinforced with horizontal reinforcement carry much higher load than the other configurations of reinforcement tried in this test.

4.6 FINITE ELEMENT ANALYSIS

Numerical analyses of the UCS tests were accomplished by PLAXIS finite element software. Elasto- plastic behavior of stone column was modeled by Mohr-Coulomb material. The geogrid was modeled as elasto-plastic continuum element. The tensile modulus (EA) of the geogrids and the yield strength are used to define the elasto-plastic behavior of geogrid. Axisymmetric finite element analysis was carried out. Mesh generation was done using 15 noded triangular elements. The nodes at the left vertical boundary were not allowed to displace horizontally but allowed to undergo vertical displacement where as for the nodes at the bottom surface, both horizontal and vertical displacements were restricted. E, υ, ϕ and Ψ for stone was taken as 30,000 kN/m^2/m, 0.3, 45˚ and 10˚respectively.

4.6.1 Comparison of load-settlement behavior for SC having slenderness ratio of 0.5, 1, 2, 3 and 4 with FEM result

UCS test was conducted on SC having slenderness ratio of 0.5, 1, 2, 3 and 4 at 30% relative density of compacted stone mass. The results are compared with PLAXIS results and found in good agreement with that.

4.6.2 Comparison of load-settlement behavior for SC reinforced with PVC and GI strip at various spacing

Fig-4.41 shows the load settlement curve of SC (l/d=2) reinforced with PVC, obtained from PLAXIS. It is observed from the fig 4.41 that the load settlement curve obtained from PLAXIS has same trend as obtained from the experimental data. Experimental results are in good agreement with PLAXIS results.

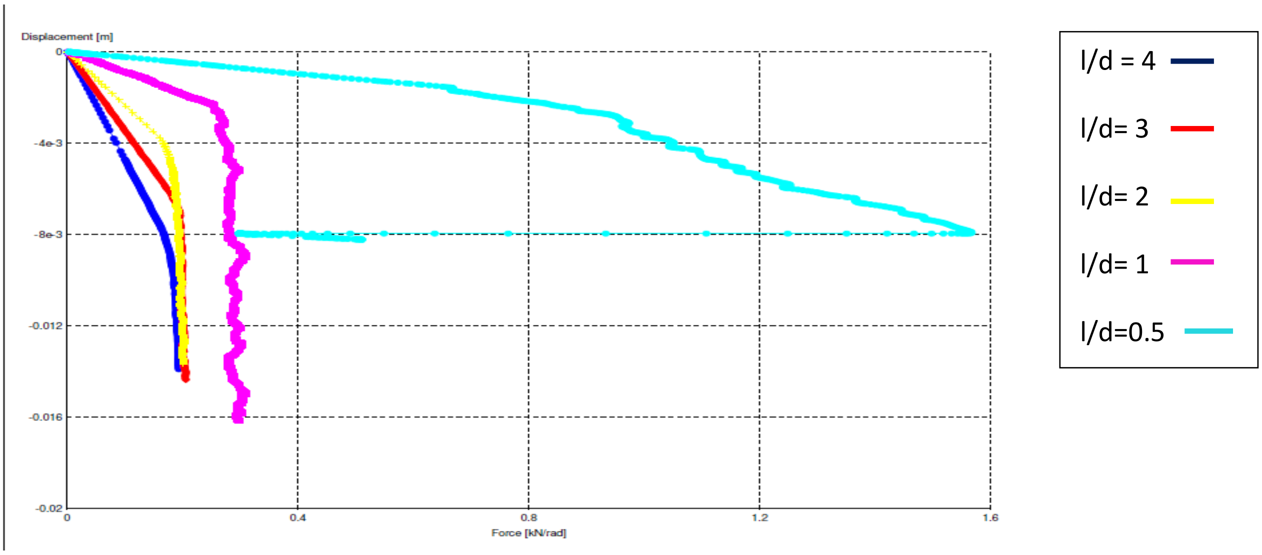
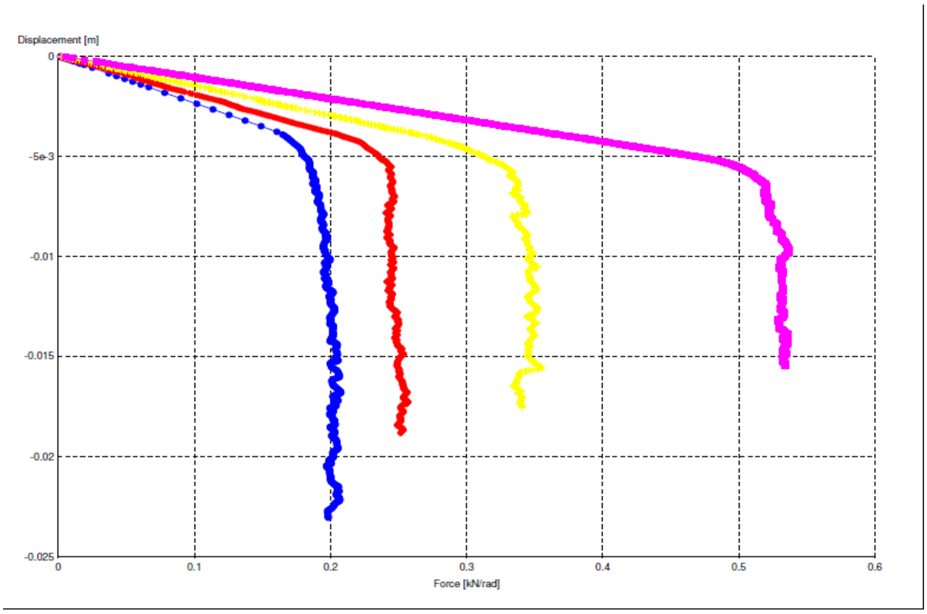


Fig-4.40 Load settlement curve of SC obtained from PLAXIS



no PVC strip PVC strip at d spacing PVC strip at 0.5d spacing PVC strip at 0.25d spacing

Fig-4.41 Load settlement curve of SC (l/d=2) reinforced with PVC, obtained from PLAXIS

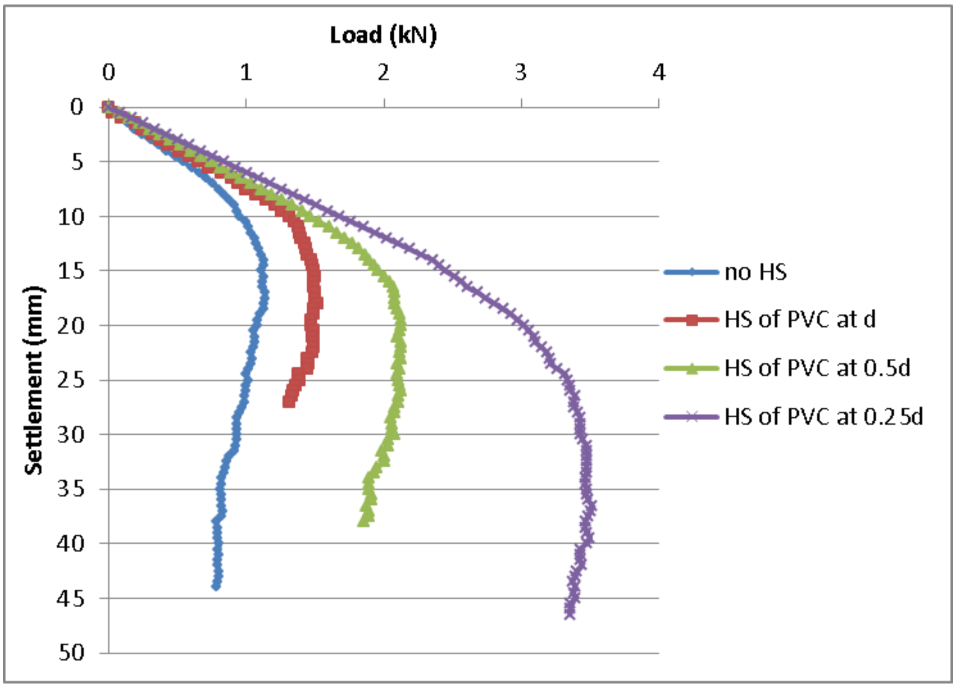


Fig-4.42 Load settlement curve of SC (l/d=2) reinforced with PVC, obtained from experimental data

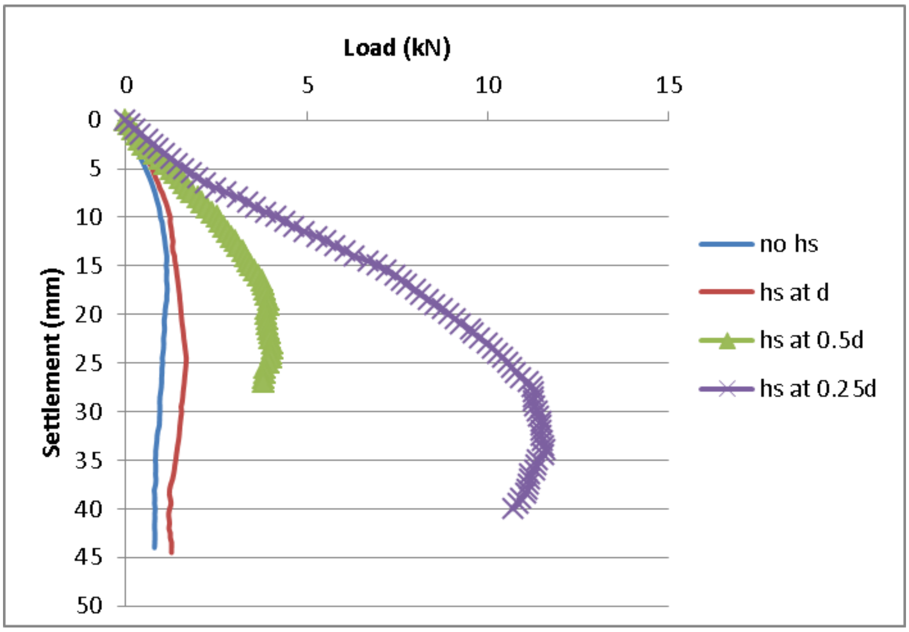


Fig-4.43 Load settlement curve of SC (l/d=2) reinforced with GI, obtained from experimental data

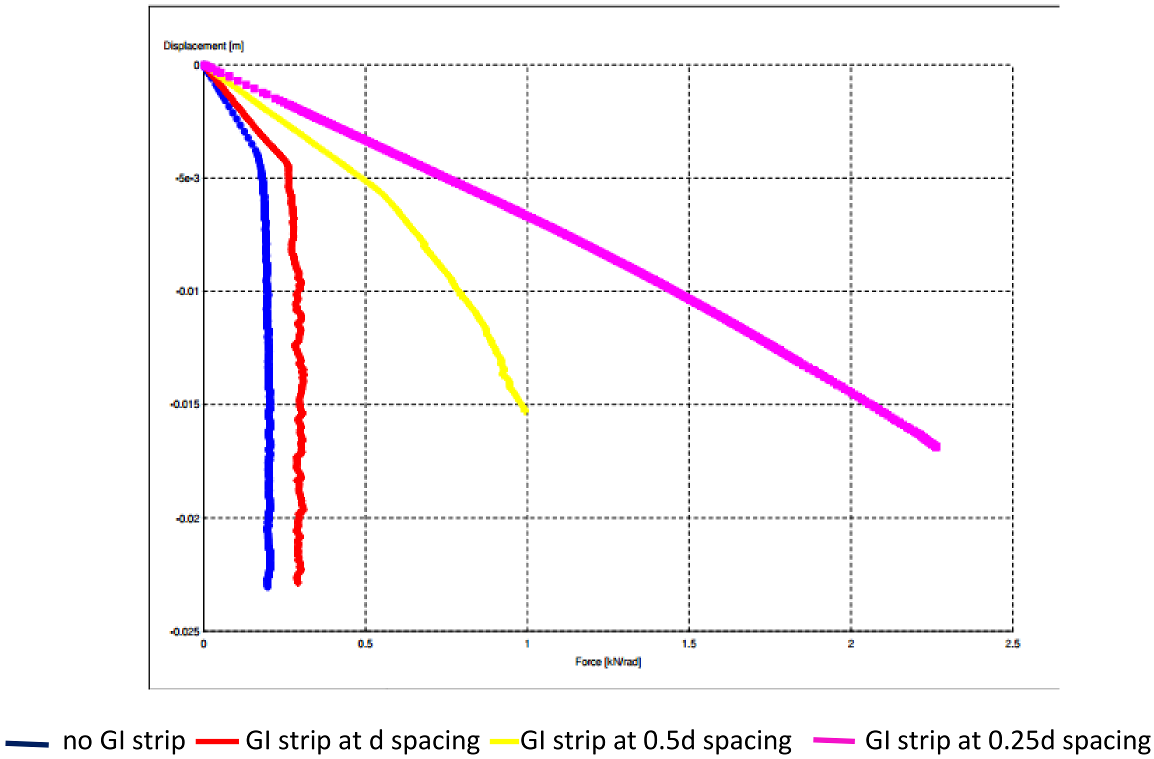


Fig-4.44 Load settlement curve of SC (l/d=2) reinforced with GI, obtained from PLAXIS data

From the fig-4.43 and 4.44 it is observed that the load settlement curve of SC reinforced with GI has similar trend for both experimental and PLAXIS results. A comparison was done on ultimate load carried by SC acquired from experimental and PLAXIS results and presented in table-4.9.

A good agreement is found to exist between the failures loads found experimentally with the numerical results. However the displacement at ultimate load is found much higher in numerical analysis than that of experimental results. This is may be due to the shortcoming in modeling of reinforcement.

Table-4.9 Comparison of ultimate load taken by SC obtained from experimental and PLAXIS

Type of SC	$(\text{Ultimate load})_{\text{Experimental}}$ in kN	$(\text{Ultimate load})_{\text{PLAXIS}}$ in kN
SC with no strip	1	1.23
SC with PVC strip at d spacing	1.5	1.5
SC with PVC strip at 0.5d spacing	2.1	2.1
SC with PVC strip at 0.25d spacing	3.5	3.3
SC with GI strip at d spacing	1.39	1.88
SC with GI strip at 0.5d spacing	3.96	6.1
SC with GI strip at 0.25d spacing	11.3	14.1

CHAPTER- 5

CONCLUSION

CONCLUSION

- There is gradual increment in load carrying capacity of ESC with increase in relative density of compacted stone aggregates. Higher relative density of the stone mass makes the SC stiff and it withstands a higher load.
- Load carrying capacity of SC decreases with increase in slenderness ratio. As the slenderness ratio changes from 2 to 4, there is a minor change in load carrying capacity of SC.
- Placement of circular strip reinforcement increases the strength of SC. For the present test variable GI strip are found to be more effective than PVC strip. Further the improvement in load carrying capacity is more pronounced if the relative density of stone mass is higher. Circular GI strip placed at a spacing of 0.25d with relative density of stone mass 90% enhance the load carrying capacity by 11.44 times over the ESC.
- SC having slenderness ratio of 0.5 fails due to rupturing of encasement, whereas SC with slenderness ratio of 1 and 2 fails mainly due to bulging effect and buckling is the main mode of failure of SC having slenderness ratio of 3 and 4. In the present test condition few ESC failed due to buckling of specimen which might be partially due to eccentric loading.
- Pond ash being highly compressible has low load bearing capacity and it can be improved by the installation of stone column and encased stone column.
- SC inserted in pond ash bed is found to fail mostly by bulging of stone mass. This bulging is observed in top 2d length. Placement of horizontal reinforcement in this region is found to be more effective than placing it beyond this region.
- Load carrying capacity of pond ash can be improved substantially with the placing of circular GI strip in the encased stone column. Insertion of GI strip in the upper 2d (d

is the diameter of SC) zone carries more load than PVC strips placed at the same region.

- Maximum bulging is found at a depth of 1.06d and its diameter at failure 1.28d for un encased SC however for ESC these values are d and 1.2d respectively, where d is the diameter of SC.
- Full encasement of SC reduces the bulging diameter and it carries more load than ordinary SC, embedded in pond ash bed. Further ESC reinforced with horizontal reinforcement carry much higher load than the other configurations of reinforcement tried in this test.
- The deformed shape of SCs exhumed after load test shows bulging of stone mass, however placement of horizontal GI strip arrests the bulging at the aggregate-reinforcement interface. Practically no bulging is found in the plane where the GI nets were provided but the PVC nets were unable to arrest the bulging. The bulging shape of SC reinforced with PVC strip is similar to that of ESC without horizontal reinforcement, which indicates that PVC strip are not effective in arresting the bulging. This may be due to lower stiffness value of PVC nets.
- A numerical analysis of encased SC and ESC with horizontal strip was examined using PLAXIS software. The stone mass was modelled as Mohr-coulomb material and the reinforcement as elasto-plastic material. A good agreement is found to exist between the failures loads found experimentally with the numerical results. However the displacement at ultimate load is found much higher in numerical analysis than that of experimental results. This is may be due to the shortcoming in modelling of reinforcement and a better model for reinforcement may improve this result.

CHAPTER -6

SCOPE FOR FUTURE STUDIES

SCOPE FOR FUTURE STUDIES

- Slenderness ratio of SC can be increased beyond 4 so as to simulate the behaviour of SC with field condition. Further the diameter and aggregate size can be enlarged to simulate the actual field condition. Position of circular reinforcement can be changed in the encased SC.
- Diameter of SC, loading area on SC, slenderness ratio, degree of saturation of ash bed, partial encasement length over the SC and aggregate size can be modified for the SC inserted in pond ash bed.
- Load test on group of SC can be made and a simple design procedure for SCs can be developed for field engineers.

CHAPTER -7

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